# AGENDA

| | |
|---|---|
| 9:00 | **INTRODUCTIONS**<br>(Bob Bea) |
| 9:15 | **PROJECT PLAN REVIEW AND UPDATE**<br>(Bob Bea) |
| 10:00 | **BREAK** |
| 10:15 | **PROJECT PROGRESS AND STATUS**<br>(Mehrdad Mortazavi) |
| 11:30 | **DISCUSSION** |
| 12:00 | **LUNCH** |
| 1:00 | **VERIFICATION CASE STUDIES**<br>(Ken Loch & Mehrdad Mortazavi) |
| 2:30 | **A COMPARISON OF LEVELS 2, 3 AND 4 OF SCREENING** (Peter Young) |
| 3:00 | **DISCUSSION / BREAK** |
| 3:15 | **SOFTWARE DEMONSTRATION**<br>(Mehrdad Mortazavi) |
| 3:45 | **PLANS FOR NEXT 6 MONTHS & MEETING** |
| 4:00 | **DISCUSSION** |
| 4:30 | **ADJOURN** |

# PROJECT SPONSORS

**Arco Exploration and Production Technology**

**California State Lands Commission**

**Exxon Production Research Company**

**Mobil Research and Development Company**

**Shell Oil Company**

**Unocal Corporation**

**&**

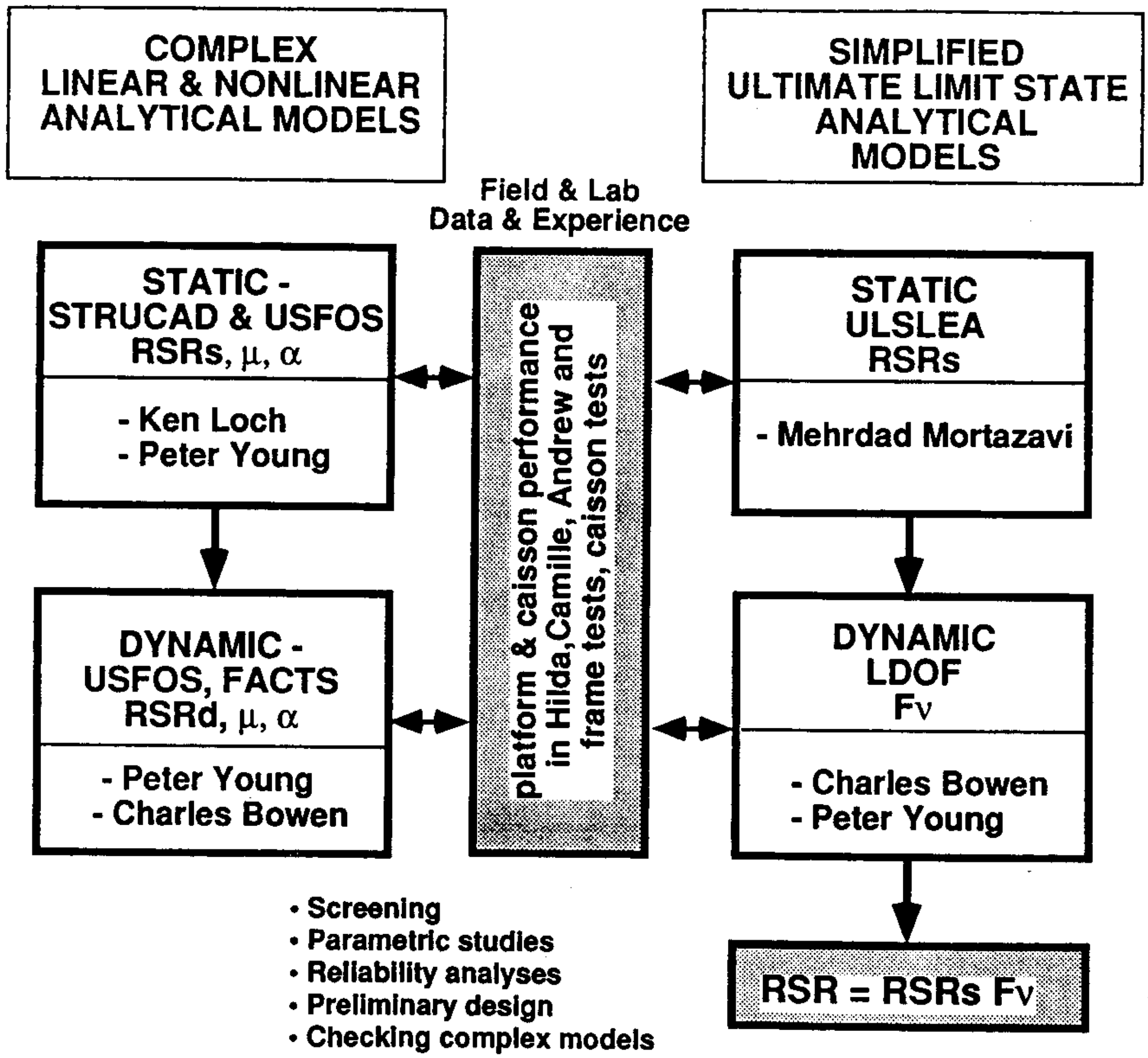
**U. S. Minerals Management Service**

**sponsoring associated projects:**

**"Verification of Screening Procedures"**

**&**

**"Dynamic Nonlinear Response In Severe Sea States"**


COMPLEX LINEAR & NONLINEAR ANALYTICAL MODELS
SIMPLIFIED ULTIMATE LIMIT STATE ANALYTICAL MODELS
Field & Lab Data & Experience
STATIC - STRUCAD & USFOS RSRs, μ, α
- Ken Loch
- Peter Young
platform & caisson performance in Hilda, Camille, Andrew and frame tests, caisson tests
STATIC ULSLEA RSRs
- Mehrdad Mortazavi
DYNAMIC - USFOS, FACTS RSRd, μ, α
- Peter Young
- Charles Bowen
DYNAMIC LDOF Fv
- Charles Bowen
- Peter Young
RSR = RSRs Fv
• Screening
• Parametric studies
• Reliability analyses
• Preliminary design
• Checking complex models

# Screening Methodologies for Use in Platform Assessments and Requalifications

## *PROJECT OBJECTIVE*

**Further develop and verify qualitative and simplified quantitative screening methodologies for platform assessments so they can be used in practice**

**... and develop a reliable and easy to use tool to check the results of level 3 and 4 platform analyses!!**

**Level 1 - 'Scoring Factors'**

**Level 2 - 'Limit Equilibrium'**

# PROJECT SCOPE REVIEW

## *LEVEL 1*
**(If time available)**

- Qualitative ranking factors and detailed assessment guidelines
- Verification / demonstration of applications

## *LEVEL 2*

- Automated input for 4, 6, 8, and 12 leg geometries
- Storm loading algorithms (shallow water, 20 th Edition procedures, loading effects)
- Element capacity modifications (joints, local wave forces, deck leg P-$\Delta$, leg capacity, pile axial failure mode, biases)
- Damaged elements (holes, dents, cracks), and repaired elements (grouted)
- Reliability analysis
- Verification / demonstration of applications

# PROJECT SCHEDULE REVIEW

## 2 years

*May 1993 - April 1994*

- LEVEL 2 - 6, 8, 12 leg geometry (Completed)
- LEVEL 2 - Element capacity modifications (Completed)
- LEVEL 2 - Verification cases (Completed)
- LEVEL 2 - Loading modifications (Started)

*May 1994 - April 1995*

- LEVEL 1 - Ranking factors & assessment guidelines (If time available)
- LEVEL 2 - Damaged & repaired elements (Started)
- LEVEL 1 & 2 Verification cases & documentation (Started)
- LEVEL 2 - Reliability analysis (Started)
- Project final report & software documentation

# DELIVERABLES

#1 - (Level 1 and) Level 2 PC code (source, IBM 486)

theory, user, and applications manuals

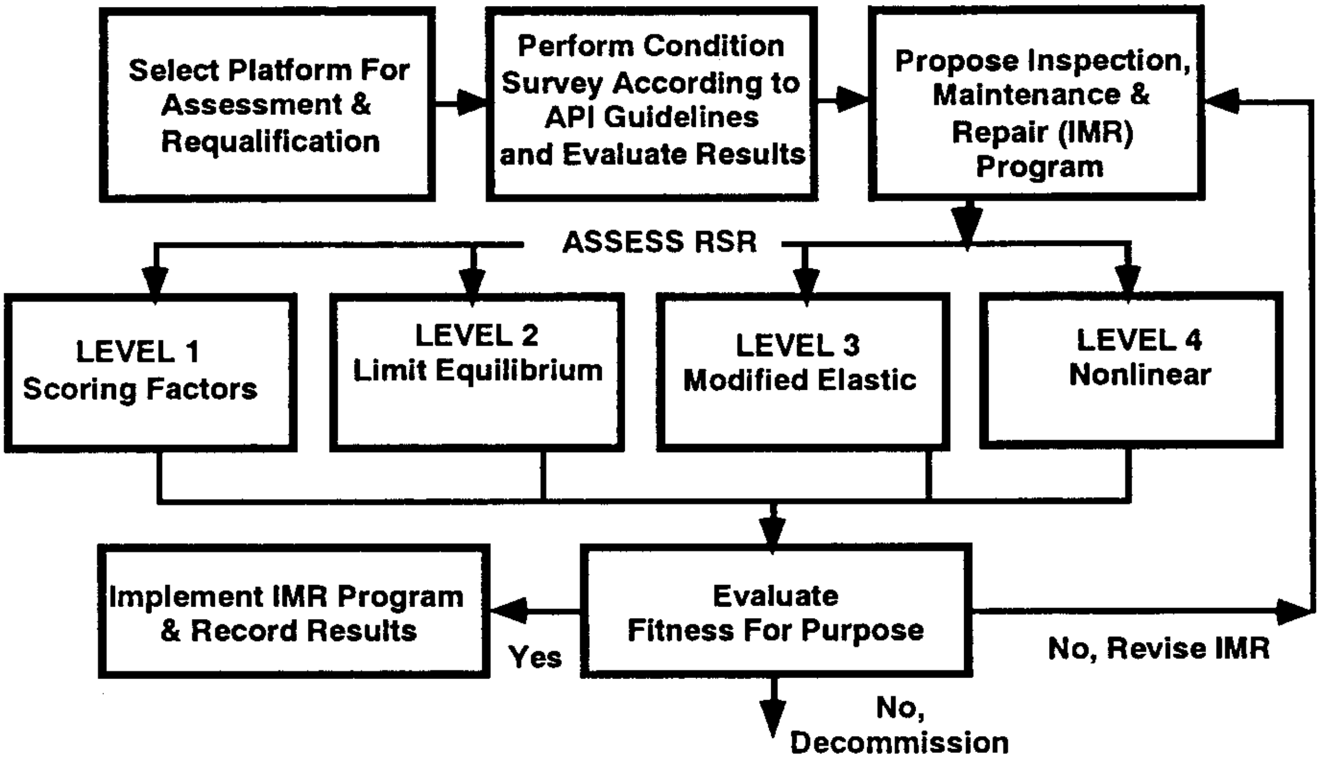
#2 - Engineering reports

background, approaches, analytical procedures, verifications

#3 - Project meetings (every 6 months)

meeting notes - project progress

# PLATFORM REASSESSMENT & REQUALIFICATION PROCESS

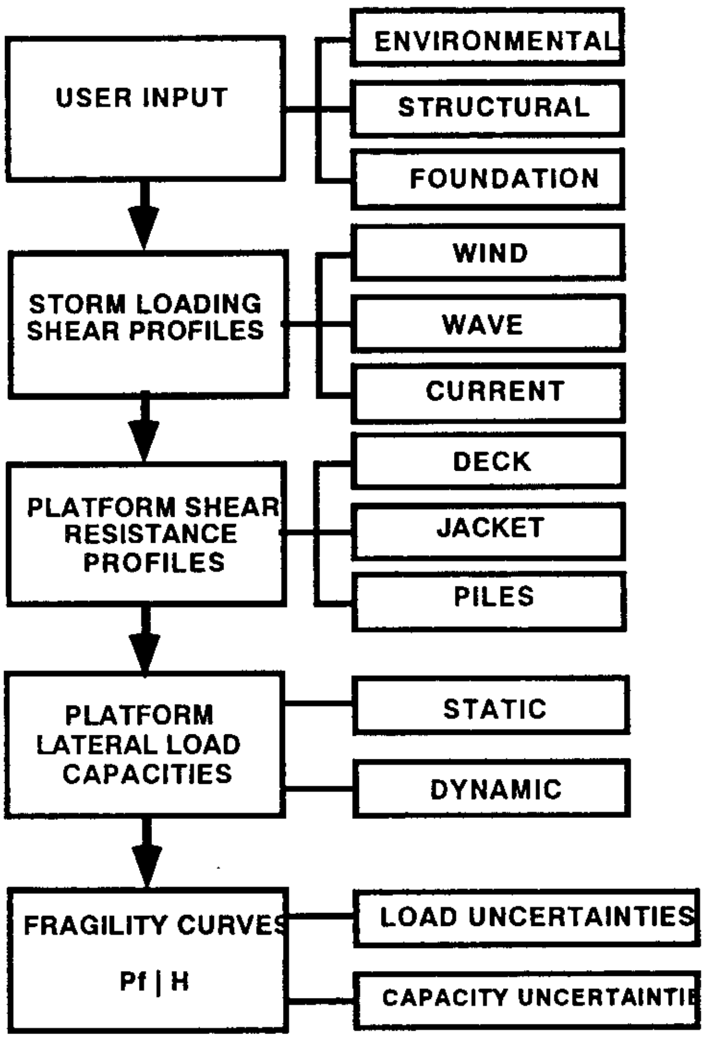


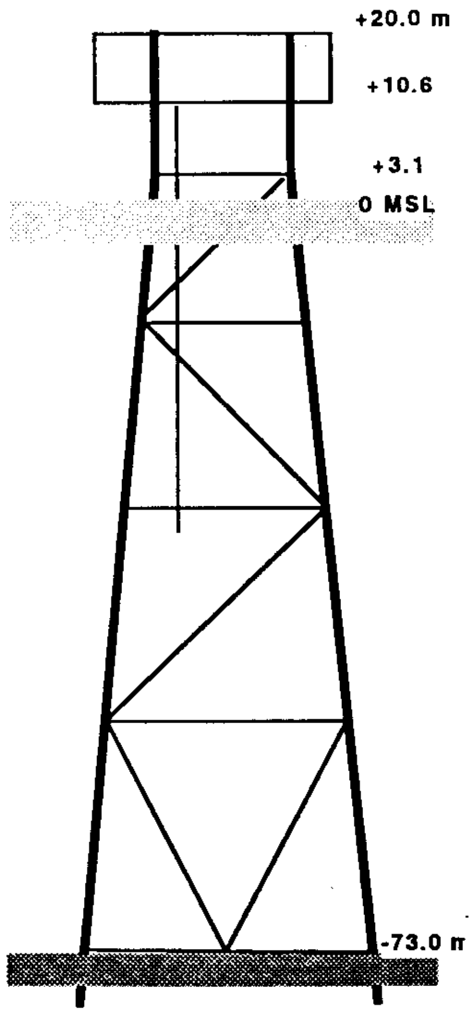

$$RSR = \frac{Ru}{Sr}$$

$$Ru = Rus\ F\ \nu$$

# LEVEL 2

# RSR 'SIMPLIFIED ANALYSES'

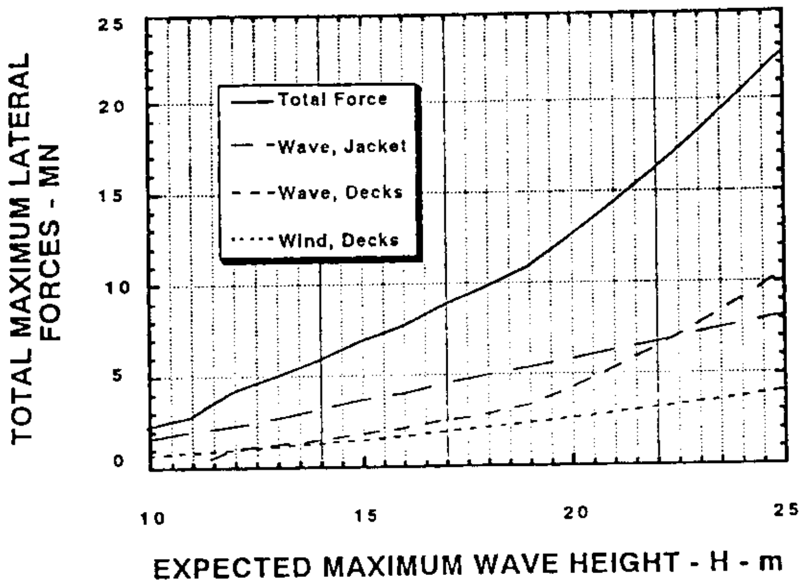


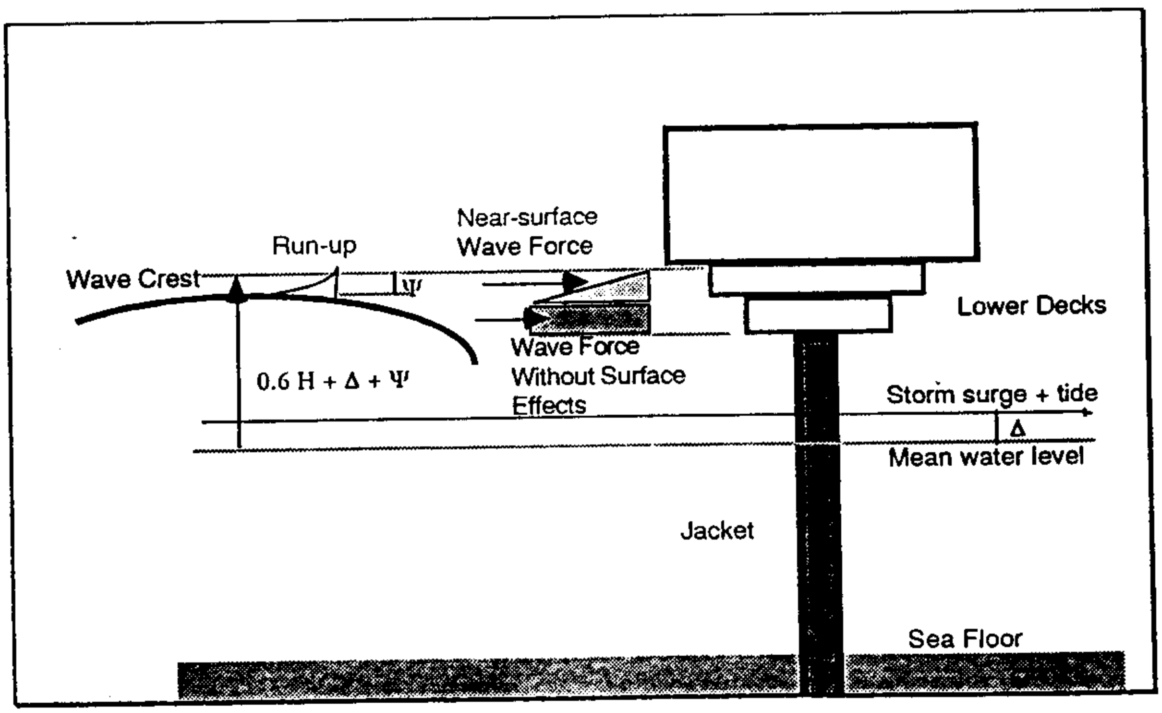



# PROJECT PROGRESS & STATUS

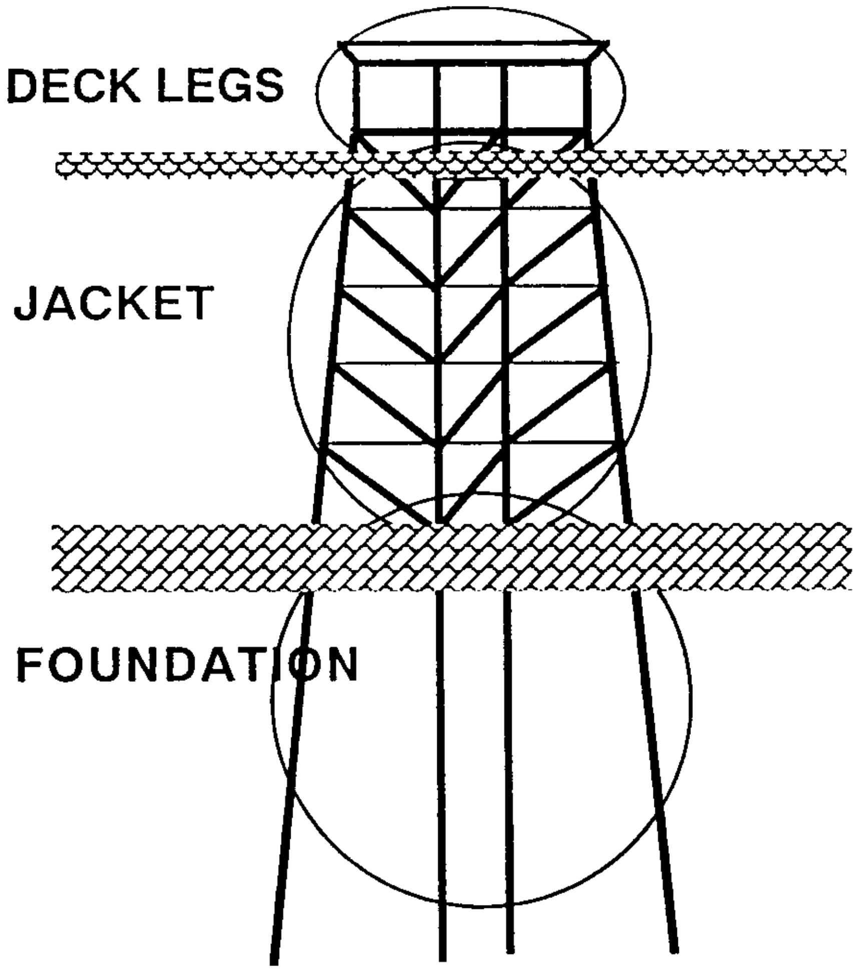
- A Review of Past Developments
- Improved Axial Compression Capacity Formulation
- Bending Moment Resistance of Jacket Legs
- Damged and Repaired Members
- Simplified Probabilistic Failure Analysis

# STORM LOADINGS

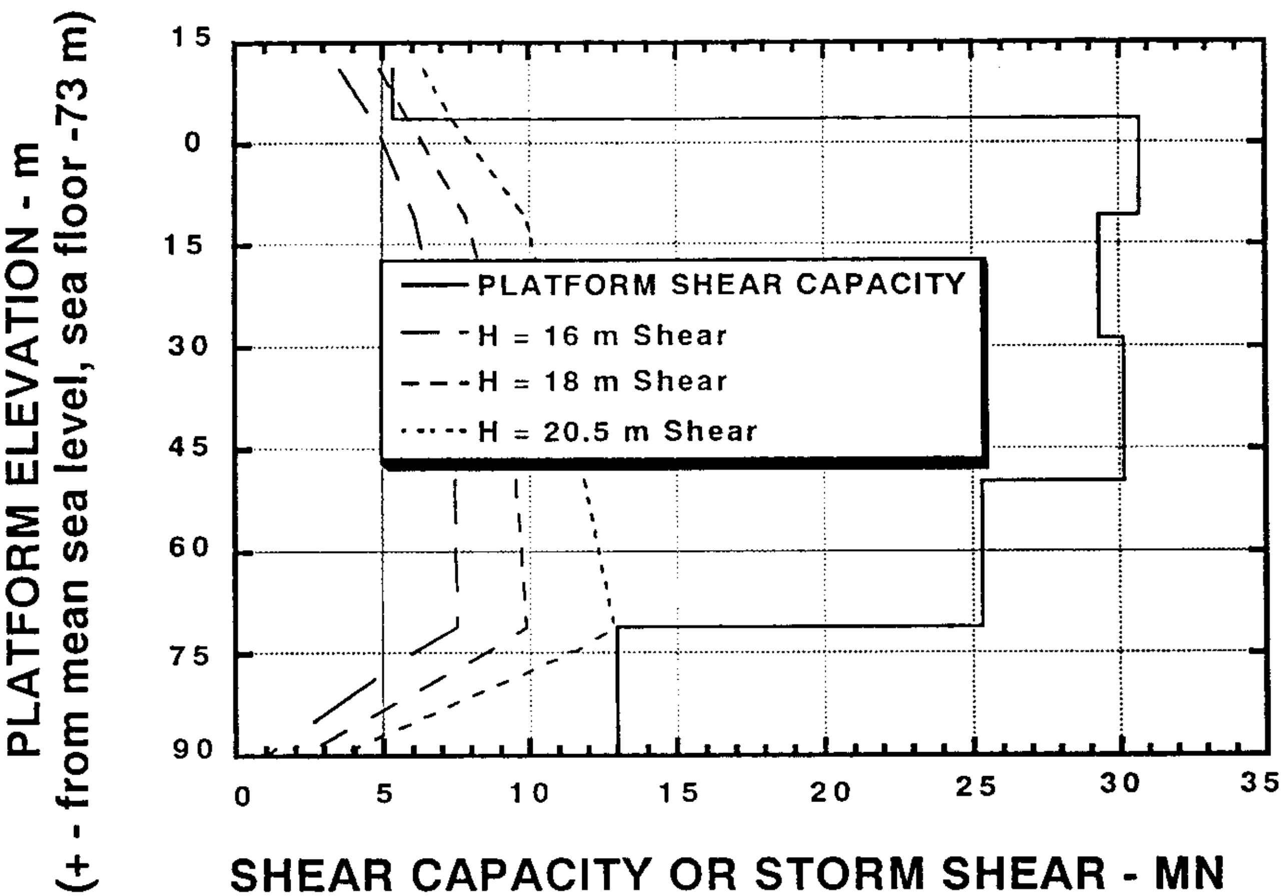


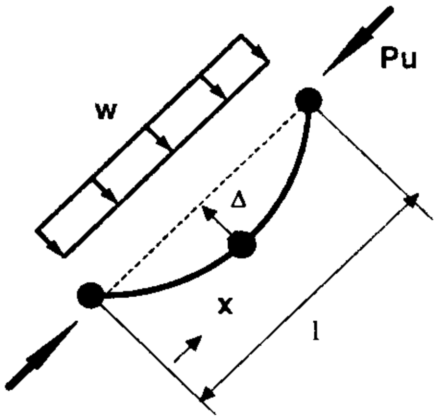
# PLATFORM CAPACITIES

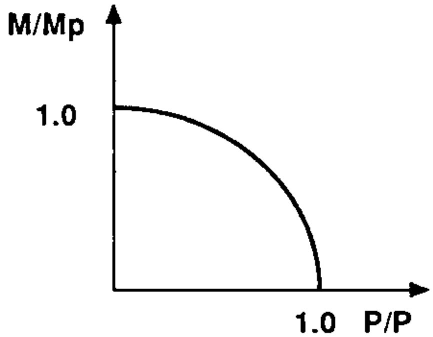


# OUTPUT


PLATFORM ELEVATION - m
(+ - from mean sea level, sea floor -73 m)
15
0
15
30
45
60
75
90
0
5
10
15
20
25
30
35
SHEAR CAPACITY OR STORM SHEAR - MN
PLATFORM SHEAR CAPACITY
H = 16 m Shear
H = 18 m Shear
H = 20.5 m Shear

# IMPROVED AXIAL CAPACITY FORMULATION
## ORIGINALLY PROPOSED APPROACH




## EQUILIBRIUM AT COLLAPSE

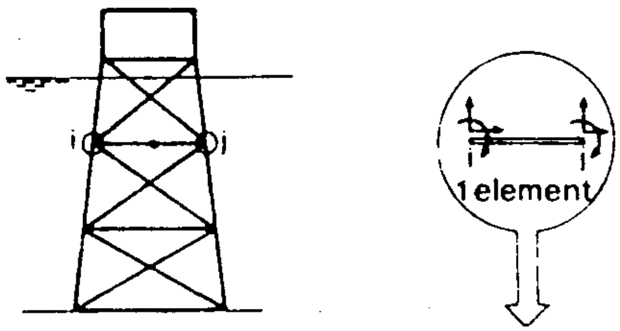
$$P_U - 2\frac{P_y}{\pi}\cos^{-1}\left(\frac{M_w + P_U\dfrac{\Delta}{1-\dfrac{P_U}{P_E}}}{M_P}\right) = 0$$

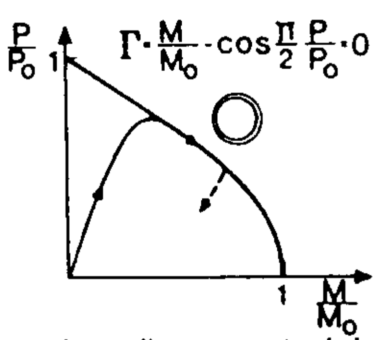
$$P_E = \frac{\pi^2 EA}{\left(Kl/r\right)^2}$$

$$\Delta = \Delta_0 + \frac{5}{384}\frac{wl^4}{EI}$$

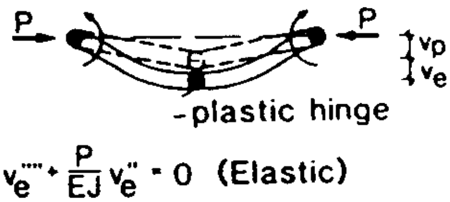
$$M_w = \frac{wl^2}{10}$$

# IMPROVED AXIAL CAPACITY FORMULATION
## USFOS APPROACH

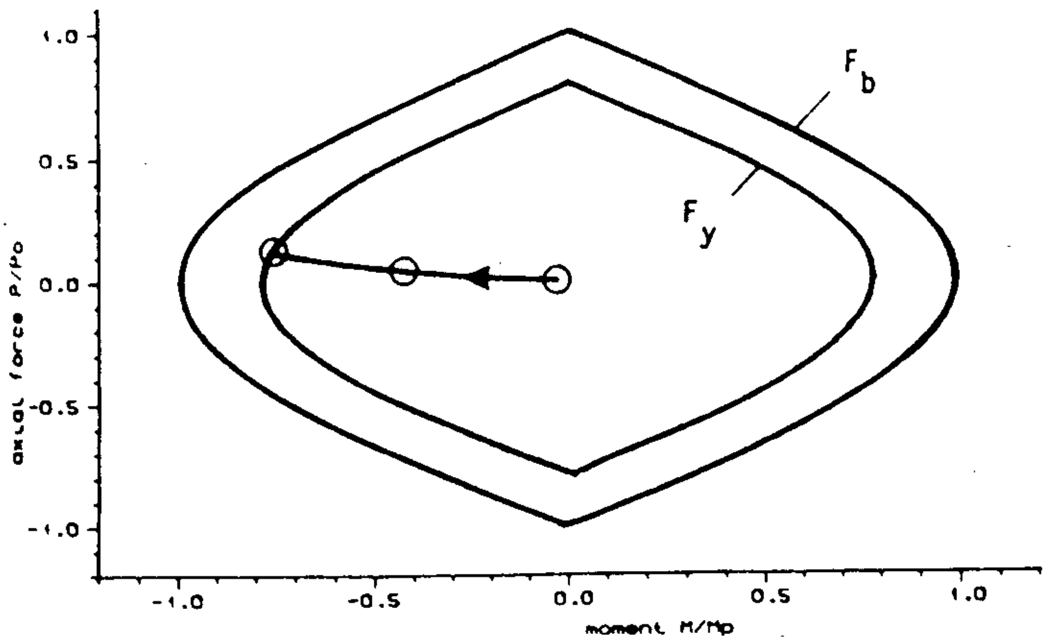


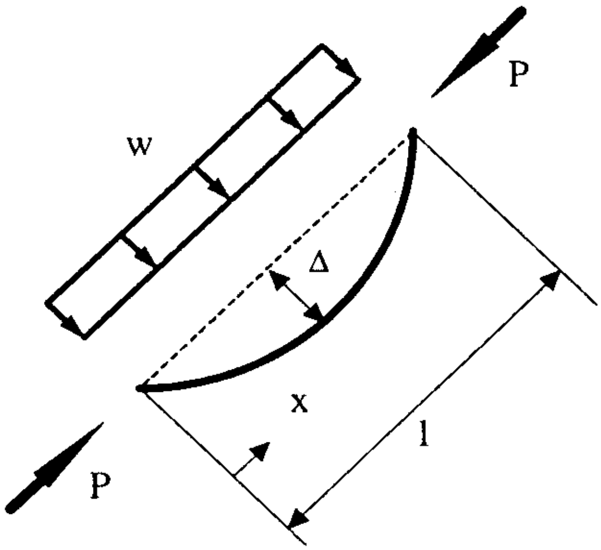

Non-linear material




Non-linear geometry



# IMPROVED AXIAL CAPACITY FORMULATION
## IMPROVED SIMPLIFIED APPROACH



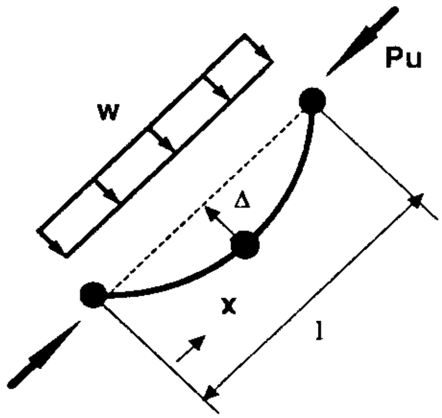

$$M_{xx} + \frac{P}{EI} M = -w - 8P\frac{\Delta_0}{l^2}$$

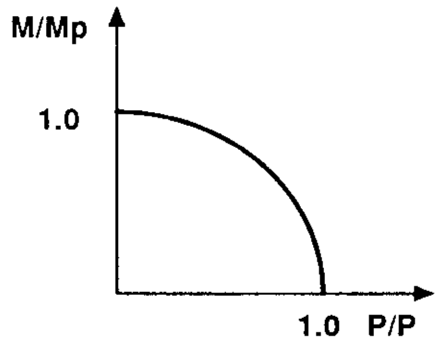
$$\xi = \frac{x}{l} \quad , \quad \varepsilon = l\sqrt{\frac{P}{EI}}$$

$$M_{\xi\xi} + \varepsilon^2 M = -wl^2 - 8P\Delta_0$$

$$M(\xi) = \frac{\sin\varepsilon(1-\xi)}{\sin\varepsilon} M(\xi=0) + \frac{\sin\varepsilon\xi}{\sin\varepsilon} M(\xi=1) + \frac{1}{\varepsilon^2}\left(\frac{\cos\varepsilon(0.5-\xi)}{\cos\frac{\varepsilon}{2}} - 1\right)\left(wl^2 + 8P\Delta_0\right)$$

# IMPROVED AXIAL CAPACITY FORMULATION
## IMPROVED SIMPLIFIED APPROACH




**EQUILIBRIUM AT COLLAPSE**

$$M(\xi = 0.5) = -M(\xi = 0) = -M(\xi = 1) = M_U$$

$$M_U = \left( \frac{1}{1 + 2\dfrac{\sin 0.5\varepsilon}{\sin \varepsilon}} \right) \frac{1}{\varepsilon^2} \left( \frac{1}{\cos \dfrac{\varepsilon}{2}} - 1 \right) \left( w\, l^2 + 8 P_U \Delta_o \right)$$

$$\frac{M_U}{M_P} - \cos\left( \frac{\pi}{2} \frac{P_U}{P_P} \right) = 0$$

# IMPROVED AXIAL CAPACITY FORMULATION
## CALIBRATION TO API BUCKLING CURVE

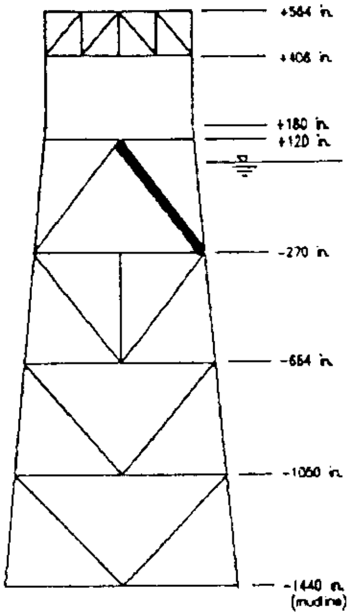
$$w = 0$$

$$P_U = P_{cr}\,(\mathrm{API})$$

## INITIAL OUT-OF-STRAIGHTNESS

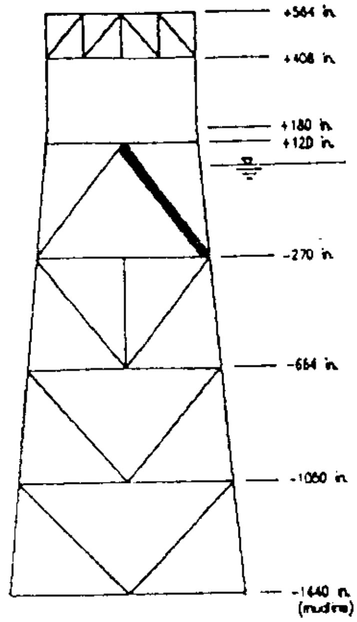
$$\Delta_0 = \frac{M_P \cos\left(\frac{\pi}{2}\frac{P_{cr}}{P_P}\right)}{\left(\frac{1}{1+2\frac{\sin 0.5\varepsilon}{\sin\varepsilon}}\right)\frac{1}{\varepsilon^2}\left(\frac{1}{\cos\frac{\varepsilon}{2}}-1\right)\left(8P_{cr}\right)}$$

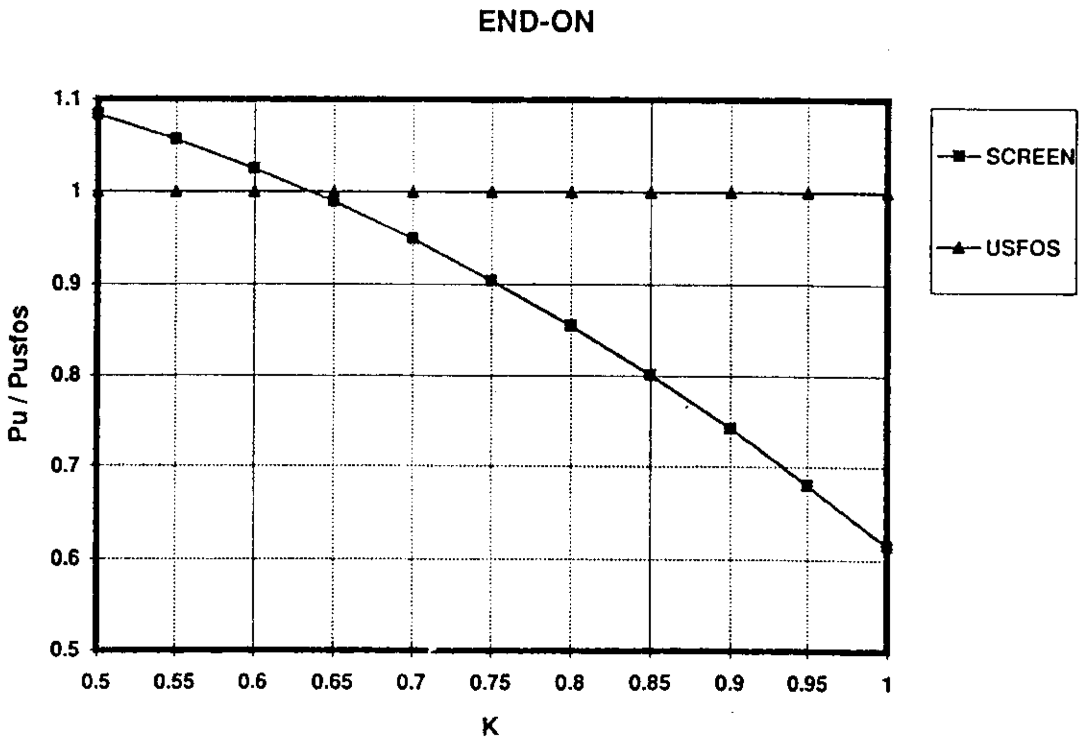
# IMPROVED AXIAL CAPACITY FORMULATION
## VERIFICATION CASES



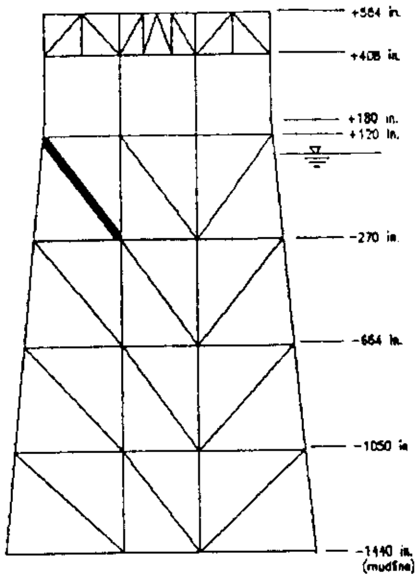

| END-ON | | | | | |
|---|---|---|---|---|---|
| METHOD | K | Delta0/L % | M KIPS-FT | Pu KIPS | Pu/Pusfos |
| USFOS | - | 0.16 | 137.66 | 607.50 | 1.00 |
| ULSLEA | - | 0.16 | 145.01 | 664.90 | 1.09 |
| ULSLEA (API) | 0.65 | 0.32 | 165.67 | 600.81 | 0.99 |

# IMPROVED AXIAL CAPACITY FORMULATION
## VERIFICATION CASES



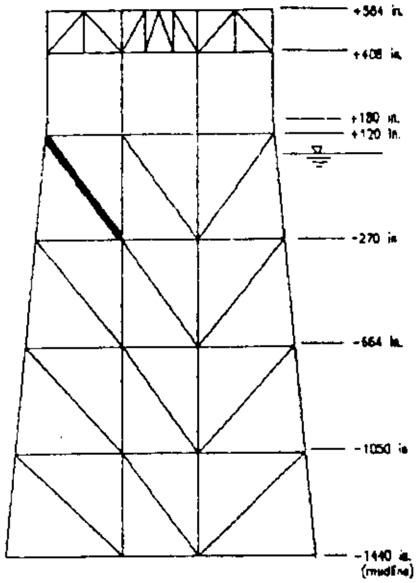



# IMPROVED AXIAL CAPACITY FORMULATION
## VERIFICATION CASES

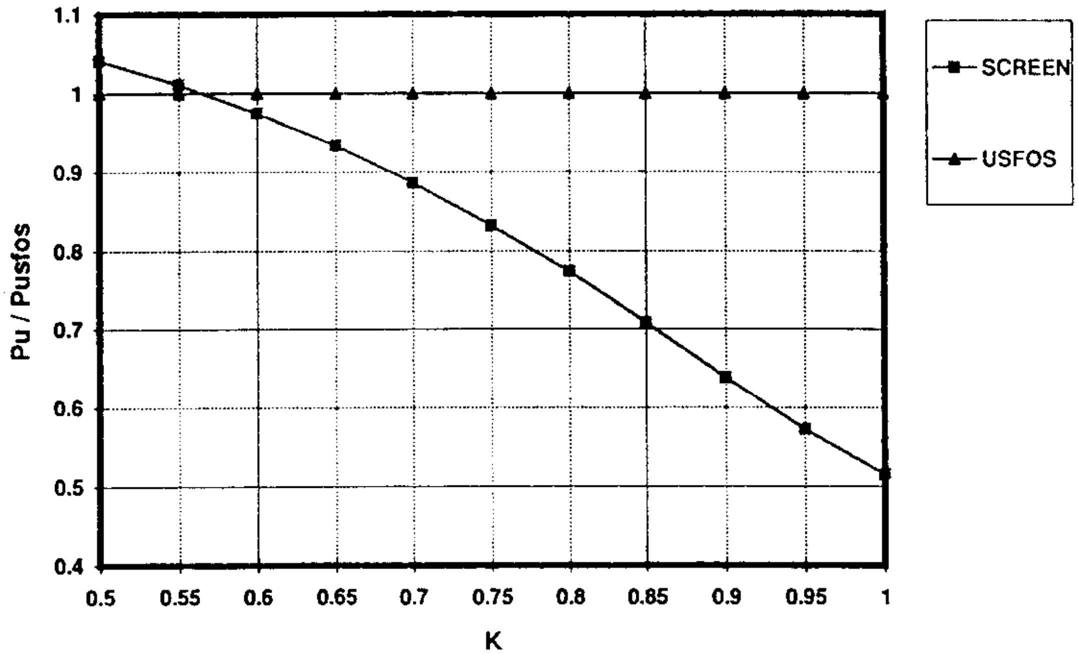



| BROADSIDE | | | | | |
|---|---|---|---|---|---|
| METHOD | K | Delta0/L<br>% | M<br>KIPS.FT | Pu<br>KIPS | Pu/Pusfos |
| **USFOS** | **-** | **0.13** | **140.08** | **611.00** | **1.00** |
| **ULSLEA** | **-** | **0.13** | **154.52** | **645.99** | **1.06** |
| **ULSLEA (API)** | **0.65** | **0.31** | **179.16** | **570.32** | **0.93** |

# IMPROVED AXIAL CAPACITY FORMULATION
## VERIFICATION CASES

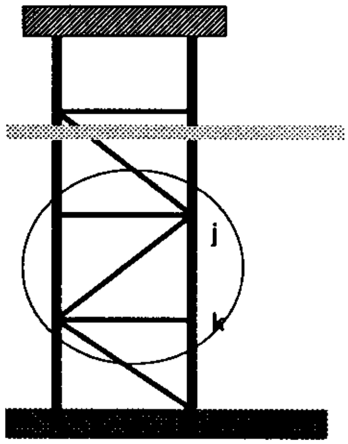


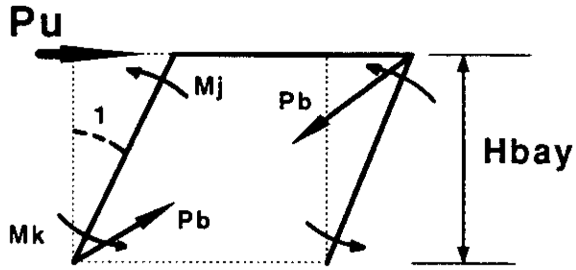

BROADSIDE



# IMPROVED JACKET CAPACITY FORMULATION

## ORIGINAL APPROACH




## VIRTUAL WORK

$$W^{(E)} = W^{(I)}$$

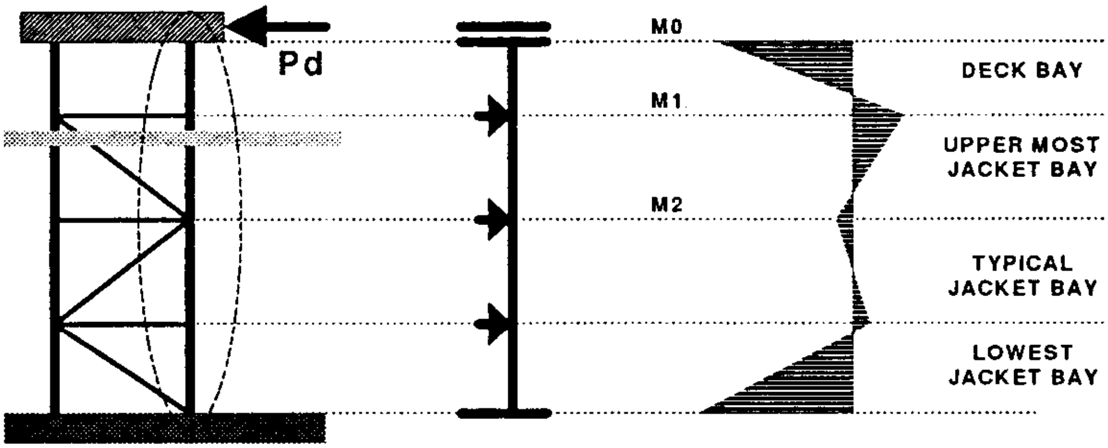
$$P_U = P_{BH} + \frac{2(M_J + M_K)}{H_{BAY}}$$

## ORIGINAL APPROACH

$$M_J = M_K = 0$$

$$P_U = P_{BH}$$

# IMPROVED JACKET CAPACITY FORMULATION
## IMPROVED APPROACH




MOMENT DISTRIBUTION IN A JACKET LEG

$$M_0 = \frac{\dfrac{P_D h_D^2}{2EI} + \dfrac{P_D h_D}{C_R}}{\dfrac{h_D}{EI} + \dfrac{1}{C_R}} \leq M_P$$

$$M_1 = M_0 - P_D h_D \leq M_P$$

**FOR EQUAL SPANS, CONSTANT MOMENT OF INERTIA AND LIMITING CASE OF RIGID SUPPORTS:**

$$|M_2| \leq 0.286 |M_1|$$

# DAMAGED AND REPAIRED MEMBERS

## OBJECTIVE:

TO DEVELOP SIMPLIFIED METHODS TO EVALUATE THE EFFECTS OF MEMBER DAMAGE AND REPAIR ON PLATFORM RESPONSE TO EXTREME LOADINGS

## DAMAGE CLASSIFICATION:

- DENTS
- GLOBAL BENDING
- CORROSION
- FATIGUE CRACKED JOINTS

# DAMAGED AND REPAIRED MEMBERS
## DENTS AND GLOBAL BENDING

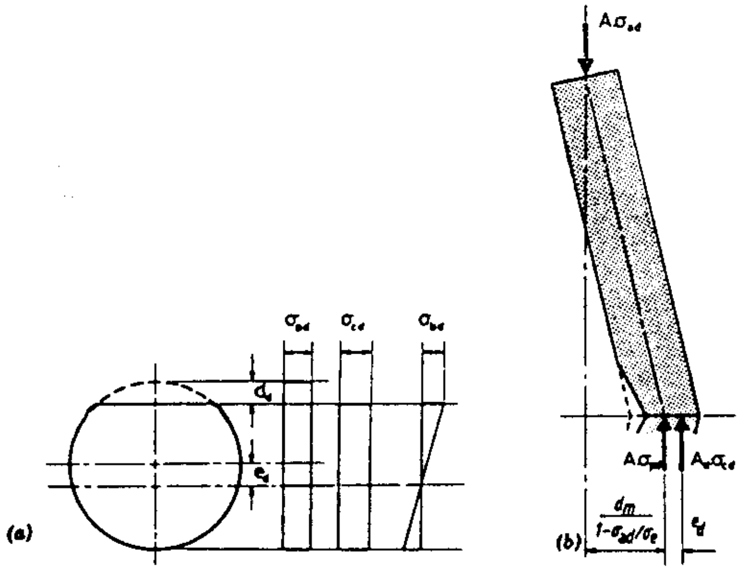
### Analytical methods

- **Beam-column Analysis** (Ellinas 1984, Chen 1987, Ricles et al. 1992, Loh 1993)

- **Numerical Integration Methods** (Kim 1992, Duan 1993)

- **Non-linear Finite Element Method (FEM)**

# DAMAGED AND REPAIRED MEMBERS
## Ellinas' Approach



$$\sigma_{pd} = \sigma_Y \frac{D}{t}\left[\sqrt{\frac{16}{9}\delta_d^2 + \left(\frac{t}{D}\right)^2} - \frac{4}{3}\delta_d\right]$$

$$\delta_d = \frac{d_d}{D}$$

$$\lambda_d = \frac{L}{r_d} - 0.2\pi\sqrt{\frac{E}{\sigma_Y}}$$

$$\frac{1}{\sigma_e}\sigma_{ud}^2 - \left[1 + \alpha\lambda_d + \frac{A_e e_d}{Z_d} + \frac{f_Y}{\sigma_e}\right]\sigma_{ud} + f_Y + \sigma_{pd}\frac{A_d e_d}{Z_d} = 0$$

# DAMAGED AND REPAIRED MEMBERS

## Loh's Unity Check Equations

### Dent-Section Capacities and Properties

$$\frac{P_{ud}}{P_u} = \frac{A_d}{A_u} = exp\left(-0.08\,\frac{dd}{t}\right) \geq 0.45 \quad , \quad \frac{M_{ud}}{M_u} = \frac{I_d}{I_u} = exp\left(-0.06\,\frac{dd}{t}\right) \geq 0.55$$

### Strength Check

$$UC = \frac{P}{P_{ud}} + \sqrt{\left(\frac{M-}{M_{ud}}\right)^{\alpha} + \left(\frac{M*}{M_u}\right)^{2}} \leq 1.0$$

$$UC = \frac{P}{P_{ud}} + \sqrt{\left(\frac{M+}{M_u}\right)^{2} + \left(\frac{M*}{M_u}\right)^{2}} \leq 1.0$$

### Stability Check

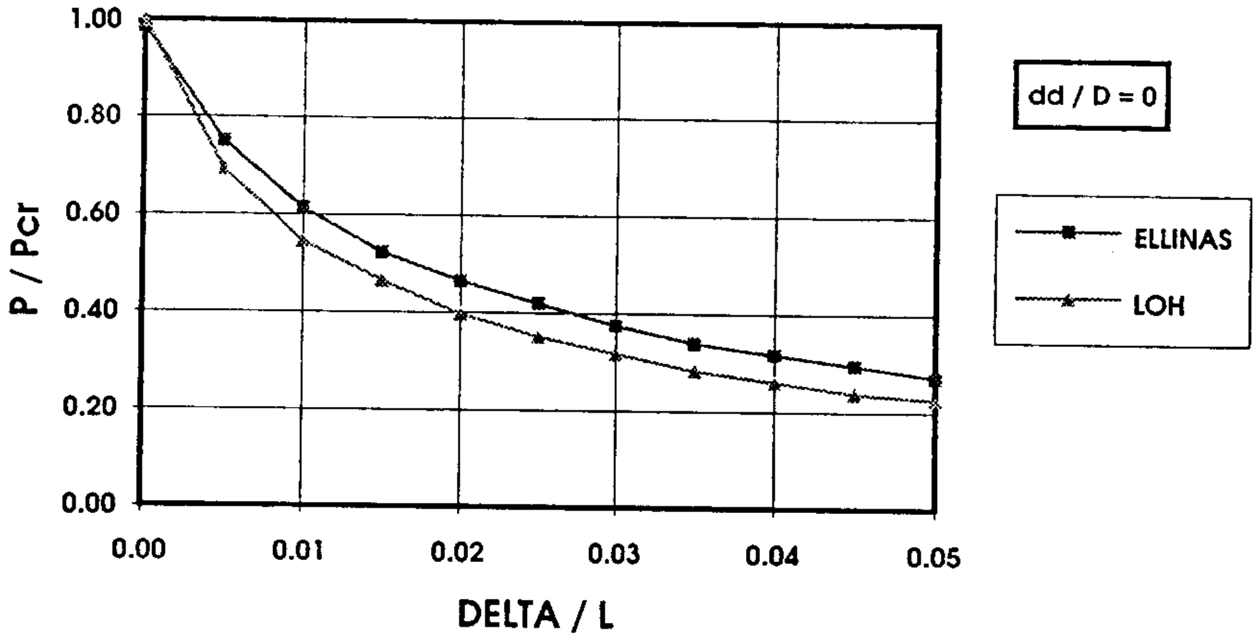
$$UC = \frac{P}{P_{crd}} + \sqrt{\left(\frac{M-}{\left(1-\frac{P}{P_{Ed}}\right)M_{ud}}\right)^{\alpha} + \left(\frac{M*}{\left(1-\frac{P}{P_E}\right)M_u}\right)^{2}} \leq 1.0$$

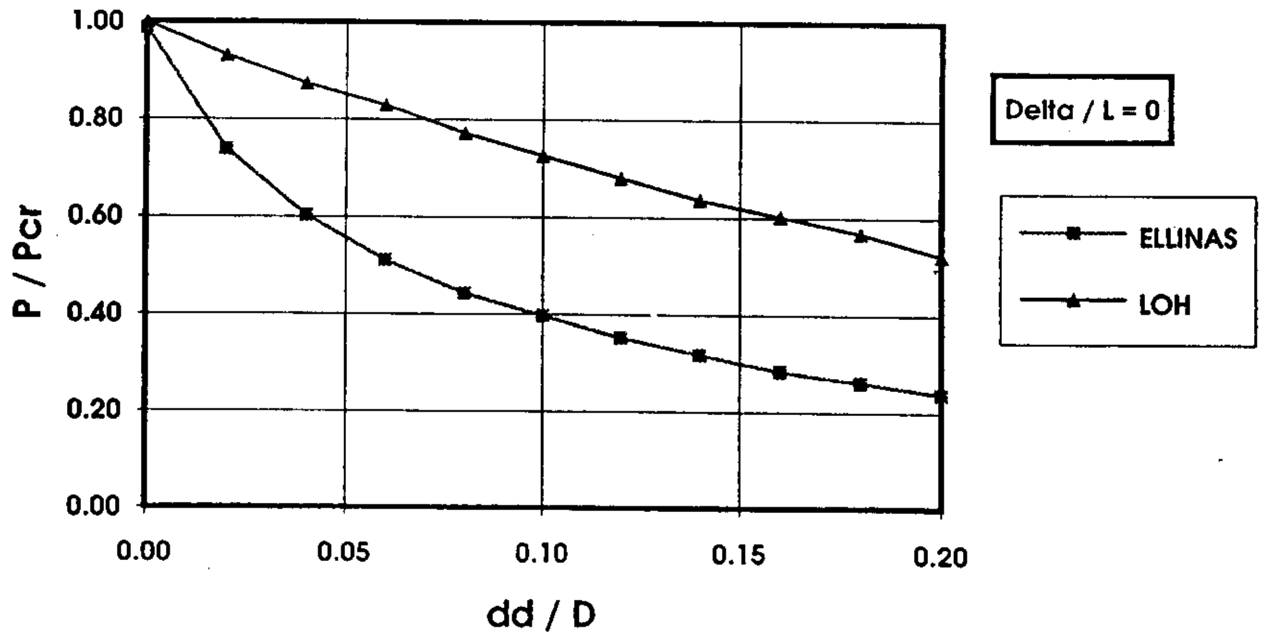
$$UC = \frac{P}{P_{crd}} + \sqrt{\left(\frac{M+}{\left(1-\frac{P}{P_{Ed}}\right)M_{ud}}\right)^{2} + \left(\frac{M*}{\left(1-\frac{P}{P_E}\right)M_u}\right)^{2}} \leq 1.0$$

$$\frac{P_{crd}}{P_{crd0}} + \frac{P_{crd}\,\Delta Y}{\left(1-\frac{P_{crd}}{P_{Ed}}\right)M_{ud}} = 1.0$$

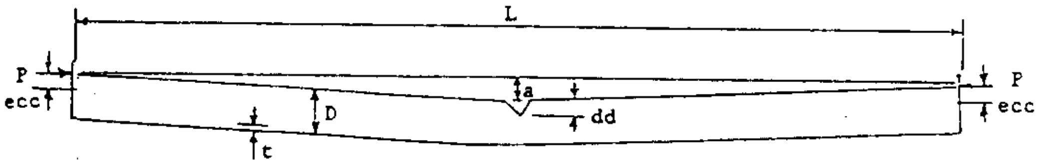
# DAMAGED AND REPAIRED MEMBERS

## Sensitivity Analysis of Ellinas' vs Loh's Formulation



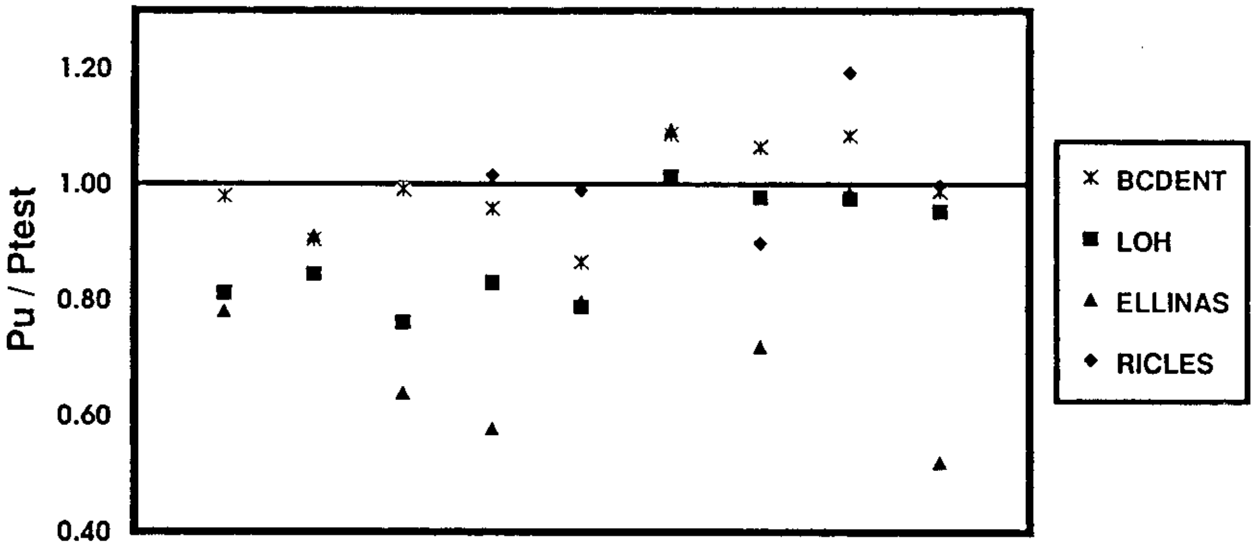
# DAMAGED AND REPAIRED MEMBERS
## A Comparison Between Experimental and Predicted Capacities




| TEST | D (IN) | t (IN) | L (IN) | Sy (KSI) | E (KSI) | dd/D (%) | delta/L (%) | e/L (%) |
|---|---|---|---|---|---|---|---|---|
| A1 | 2.50 | 0.08 | 84.63 | 33.06 | 29145 | | 0.02 | |
| A2 | 2.50 | 0.08 | 84.63 | 33.21 | 30160 | | 0.03 | 0.46 |
| A3 | 2.50 | 0.08 | 84.63 | 32.77 | 28710 | 0.046 | 0.55 | |
| B3 | 3.13 | 0.07 | 84.63 | 28.71 | 31030 | 0.08 | 0.5 | |
| C1 | 4.00 | 0.07 | 84.63 | 30.60 | 29145 | | 0.05 | |
| C2 | 4.00 | 0.07 | 84.63 | 41.18 | 29870 | | 0.05 | 0.46 |
| C3 | 4.00 | 0.07 | 84.63 | 33.79 | 28565 | 0.034 | 0.04 | |
| F1 | 16.02 | 0.39 | 305.24 | 44.23 | 28710 | | 0.07 | |
| F2 | 15.98 | 0.39 | 305.24 | 42.49 | 31030 | 0.124 | 0.18 | |

| Test | dd/D | delta/L (%) | e/L (%) | Ptest (KN) | BCDENT (KN) | LOH (KN) | ELLINAS (KN) | RICLES (KN) |
|---|---|---|---|---|---|---|---|---|
| A1 | | 0.02 | | 78.10 | 76.50 | 63.46 | 60.94 | - |
| A2 | | 0.03 | 0.46 | 46.00 | 41.60 | 38.86 | 41.88 | - |
| A3 | 0.05 | 0.55 | | 44.20 | 43.80 | 33.68 | 28.23 | - |
| B3 | 0.08 | 0.50 | | 43.30 | 41.50 | 35.97 | 25.04 | 43.96 |
| C1 | | 0.05 | | 121.00 | 104.80 | 95.37 | 96.48 | 119.66 |
| C2 | | 0.05 | 0.46 | 89.40 | 97.10 | 90.66 | 97.84 | - |
| C3 | 0.03 | 0.04 | | 95.70 | 101.90 | 93.61 | 68.85 | 86.00 |
| F1 | | 0.07 | | 3238.70 | 3509.90 | 3160.30 | 3192.10 | 3862.30 |
| F2 | 0.12 | 0.18 | | 2056.90 | 2031.70 | 1962.40 | 1068.00 | 2051.40 |

# DAMAGED AND REPAIRED MEMBERS
## A Comparison Between Experimental and Predicted Capacities



# DAMAGED AND REPAIRED MEMBERS

## Sensitivity Analyses and Conclusions

- The residual strength decreases significantly as the dent depth increases.

- Column strength is more sensitive to local denting damage when slenderness parameter $\lambda$ is small.

- For a given dent depth, the analyses show a decrease in residual strength for members with higher D/t ratio.

- There is negligible conservatism in assuming a mid-length dent location for any practical dent within the middle-half section of a members effective length.

- Lateral loadings, such as those caused by wave forces, can significantly affect dented brace capacity.

- Ricles (1993): DENTA ( developed by Taby 1988), Loh's interaction equation, numerical integration based on M-P-$\Phi$ relationships, and the non-linear FEM are able to predict the residual capacity of the test members reasonably well.

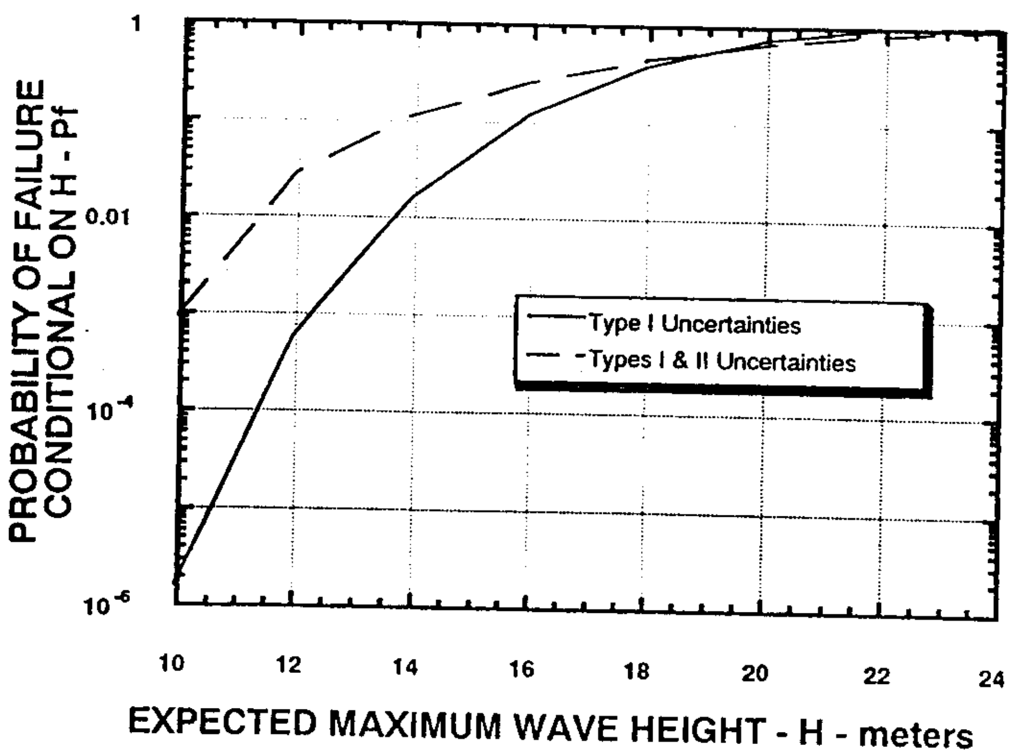
# PROBABILISTIC FAILURE ANALYSIS
## Objective and Background

### Objective:

To develop a reliability based level 2 screening procedure to identify critical platforms and their potential failure modes

### Background:

$$P_f|H = 1 - \Phi\left(\frac{ln\left(\frac{R_u}{S|H}\right)}{\sqrt{\sigma^2_{ln R} + \sigma^2_{ln S} - 2\rho\,\sigma_{ln R}\,\sigma_{ln S}}}\right)$$



# PROBABILISTIC FAILURE ANALYSIS
## FOSM Based Component and System Reliability

$$M = \ln R - \ln S$$

$$U = \frac{(M - \mu_M)}{\sigma_M}$$

$$P_f = CDF(U)$$

assuming lognormal distribution for loads and capacities the exact reliability index can be given as:

$$\beta = \frac{\mu_M}{\sigma_M}$$

$$\mu_M = \ln\left(\frac{\mu_R}{\mu_S}\sqrt{\frac{1+V_S^2}{1+V_R^2}}\right)$$

$$\sigma_M^2 = \ln(1+V_R^2) + \ln(1+V_S^2) - 2\ln(1+\rho_{RS} V_R V_S)$$

$$P_f = \Phi(-\beta)$$

**Series System:**

$$\max P_{fi} < P_{fs} < \sum_i P_{fi}$$

# PROBABILISTIC FAILURE ANALYSIS
## Uncertainty in Loading

### WAVE LOADING:

$$S_H = K_H \, H^{\alpha}$$

### Drag force dominated structure:

$$S_H = K_d \; K_u \, H^2$$

## Professor Bea:

| $x$ | $\sigma_{lnx}$ | $BIAS\ (B_x)$ | $\sigma_{lnBx}$ |
|---|---|---|---|
| $K_u$ | 0.1 | 0.41 | 0.47 |
| $K_d$ | 0.1 | 1.67 | 0.23 |
| *Hmax (G.O.M)* | 0.3 | 1.1 | 0.13 |

# PROBABILISTIC FAILURE ANALYSIS
## Uncertainty in Component Capacities

**Deck Capacity:**

$$\mu_{RD} = \mu_{Mcr} \cos[(\pi Q / 2n\, \mu_{Pcrl})](2n - QL^2/ 6EI)/L$$

$$\sigma_{RD} = [\sigma^2_{Mcr}(\delta RD/\delta Mcr)^2 + \sigma^2_{Pcrl}(\delta RD/\delta Pcrl)^2 + 2\sigma_{Mcr}\sigma_{Pcrl} (\delta RD/\delta Mcr)(\delta RD/\delta Pcrl)]^{1/2}$$

where

$$\delta RD/\delta Mcr = \cos[(\pi Q / 2n\, \mu_{Pcrl})](2n - QL^2/ 6EI)/L$$

$$\delta RD/\delta Pcrl = (\pi Q \mu_{Mcr} / 2n\, (\mu_{Pcrl})^2) \sin[(\pi Q / 2n\, \mu_{Pcrl})] (2n - QL^2/ 6EI)/L$$

$$V_{Mcr} = \underline{0.106},\ V_{Pcrl} = \underline{0.117}$$

- $V_{Mcr}$, $V_{Pcrl}$ are reported to be constant over the entire range of practical values of Et/fyD and D/t respectively.

# PROBABILISTIC FAILURE ANALYSIS
## Uncertainty in Component Capacities

### Jacket Bay Capacity:

$$\mu_{RJi} = \Sigma\, \alpha_i \mu_{Ri} + \mu_{RL}$$

$$\sigma_{RJi} = [\, \Sigma\, (\alpha_i \sigma_{Ri})^2 \; + \Sigma\, \alpha i \alpha j\; \sigma_{Ri}\sigma_{Rj} \; + (B_{FL}\sigma_{RL})^2 ]^{1/2}$$

$$R_i = f(\lambda)$$

$$\lambda = (1/\pi)\,(fy/E)^{0.5}\,(KL/r)$$

| $\lambda$ | 0.4 | 0.6 | 0.8 | 1.0 | 1.2 | 1.4 |
|---|---|---|---|---|---|---|
| $V R_i$ | 0.099 | 0.100 | 0.106 | 0.119 | 0.150 | 0.212 |

# PROBABILISTIC FAILURE ANALYSIS
## Uncertainty in Component Capacities

### Foundation Capacity:

Axial capacity:

$$\mu_{RFa} = \mu_q A_P + \mu_f A_S$$

| *Axial Pile Capacity in* | *Bias* | *C.O.V.* |
|---|---|---|
| *Sand* | *0.9* | *0.47 - 0.56* |
| *Clay* | *1.3 - 3.7* | *0.32 - 0.53* |

Lateral capacity in clay:

$$\mu_{RFl} = 1/2[-27 D^2 \mu_{Su} + (27 D^2 \mu_{Su})^2 + 144 \mu_{Su} D (\mu_{fy} - Q/nA) Z]^{0.5} + \mu_{RL}$$

Lateral capacity in sand:

$$\mu_{RFl} = 2.382 ((\mu_{fy} - Q/A_P) Z)^{2/3} (\mu_\gamma D \tan^2(45+\mu_\phi/2))^{1/3} + \mu_{RL}$$

| *Lateral Capacity in* | *Bias* | *C.O.V.* |
|---|---|---|
| *Clay* | *0.92* | *0.20* |
| *Sand* | *0.81* | *0.21* |

# PROBABILISTIC FAILURE ANALYSIS
## Example Application

| $f_{Hm}(h)$ | $\mu_{Hm}$ *(feet)* | $\sigma_{Hm}$ *(feet)* | *PAR1* | *PAR2* |
|---|---|---|---|---|
| *Lognormal* | *34.5* | *11.7* | *0.33* | *3.49* |
| *Type I largest* | *34.0* | *11.4* | *28.9* | *0.11* |

| END-ON LOADING | SHEAR (KIPS) | BIAS | C.O.V. | RESIST. (KIPS) | BIAS | C.O.V. | FOSM $\beta$ | FOSM Pf | FORM $\beta$ | SORM $\beta$ |
|---|---|---|---|---|---|---|---|---|---|---|
| DECK LEGS | 120 | 0.83 | 1.03 | 2606 | 1.00 | 0.11 | 4.22 | 1.20E-05 | 4.10 | 4.10 |
| JACKET | | | | | | | | | | |
| BAY1 | 424 | 0.83 | 1.03 | 1954 | 1.00 | 0.07 | 2.43 | 7.51E-03 | 2.29 | 2.29 |
| BAY2 | 499 | 0.83 | 1.03 | 2046 | 1.00 | 0.10 | 2.28 | 1.13E-02 | 2.12 | 2.14 |
| BAY3 | 515 | 0.83 | 1.03 | 2360 | 1.00 | 0.15 | 2.39 | 8.54E-03 | 2.25 | 2.28 |
| BAY4 | 518 | 0.83 | 1.03 | 2538 | 1.00 | 0.20 | 2.43 | 7.62E-03 | 2.28 | 2.32 |
| BAY5 | 520 | 0.83 | 1.03 | 2892 | 1.00 | 0.26 | 2.51 | 6.02E-03 | 2.38 | 2.43 |
| FOUNDATION | | | | | | | | | | |
| LATERAL | 520 | 0.83 | 1.03 | 7200 | 0.81 | 0.53 | 2.88 | 1.96E-03 | 2.85 | 2.85 |
| AXIAL | 856 | 0.83 | 1.03 | 4063 | 1.5 | 0.31 | 2.74 | 3.12E-03 | 2.70 | 2.70 |

| Bimodal Bounds | Lower Bound | Upper Bound |
|---|---|---|
| Pf | 0.038 | 0.056 |
| $\beta$ | 1.78 | 1.59 |

# PROBABILISTIC FAILURE ANALYSIS
## Summary and Conclusions

- A simplified procedure is presented to perform structural reliability analysis of conventional, steel jacket, offshore platforms, which can be used in the process of reassessment and requalification of older platforms.

- The analysis is based on a first order second moment approach.

- It is assumed that the loads and capacities are lognormally distributed.

- The results from the simplified FOSM analysis are in good agreement with those gained from more sophisticated first order and second order reliability methods (FORM and SORM).

# PROBABILISTIC FAILURE ANALYSIS
## Future Work

- Considering correlations between load and resistance
- Including the uncertainty associated with joint capacities
- Integrating the reliability analysis procedure in "ULSLEA"

# Verification of Screening Methodologies for Use in Gulf of Mexico Platform Requalifications

Kenneth J. Loch

# Project Objectives

- To further develop and verify the viability of Level 2 screening methods
- To utilize hurricane Andrew platform survival and failure experiences to help verify Level 4 non-linear analyses

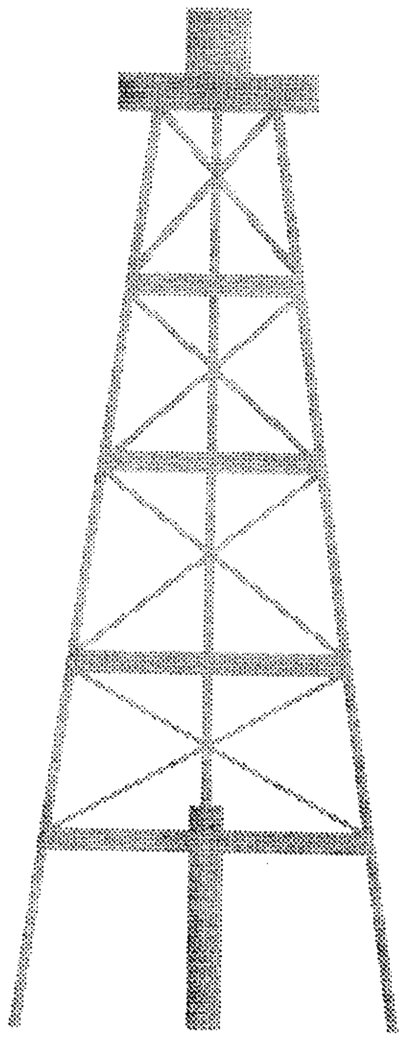
# Project Scope
# Jan. 94 - Dec. 94

- Amoco ST 161A
- PMB Benchmark
- Chevron ST 151H
- Chevron ST 151K
- Report

# Verification Case Study Status

- Amoco ST 161A - completed
- PMB Benchmark - completed
- Kerr McGee ST 34-2,3 - completed
- Kerr McGee ST 34-4 - completed
- Chevron ST 151H - data available
- Chevron 151K - data available
- Shell SP 62 - data available
- Shell SS 274 - data available
- Phillips SMI 76B - data available
- Phillips NCI - A - data available
- others (Mobil, Unocal, Exxon)

# Amoco ST 161A



# General Description

- Eight leg drilling and production platform
- Designed by McDermott using 25-year Glenn storm (H=55 ft.)
- Installed in 118 ft of water in 1964
- Broadside and end-on framing battered at 1:8
- Cellar and main decks at +34 ft and +47 ft respectively

# Platform Details

- No joint cans (0.5 in. jacket leg thickness)
- Gusset plates used for leg K-joints
- $F_y$ = 43 ksi or 58 ksi
- 36 in. piles penetrate 165 ft of soft to stiff clay and 25 ft of dense sand
- Vertical braces range in size from 14 in. in the fourth (upper) bay to 20 in. in the first bay

# Platform History

- 1972: First risk analysis performed
- 1973: Pile-leg annulus grouted as a result of assessment
- 1974: Hurricane Carmen
  - eye passed within 10 miles of platform
  - hindcast 58 ft wave, SE, no damage
- 1988: Risk analysis, all eight conductors removed, bottom deck cleared

# Platform History Continued

- 1992: Hurricane Andrew
  - eye passed within 8 miles of platform
  - 60-64 ft waves, ESE
  - yielding in +10 ft K-joints, no grout
- 1992: Risk analysis and retrofits
  - 10% more load would cause collapse
  - conductor removal reduces loads by 20%
  - +10 ft K-joints grouted

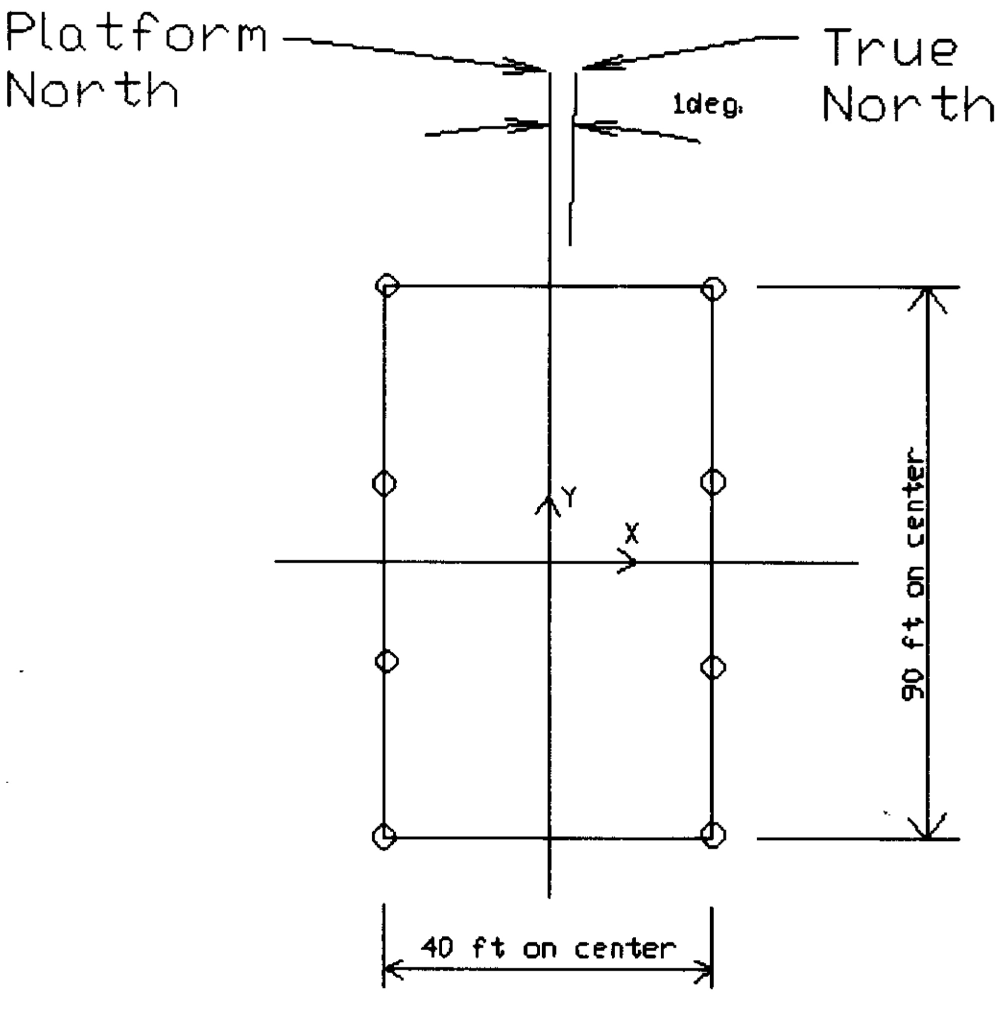
# Level 4 Analysis

- Static pushover analysis
- Utilized Amoco's 1992 USFOS model
- WAJAC generated hydrodynamic loads
- Broadside and end-on analyzed separately to match Level 2 approach

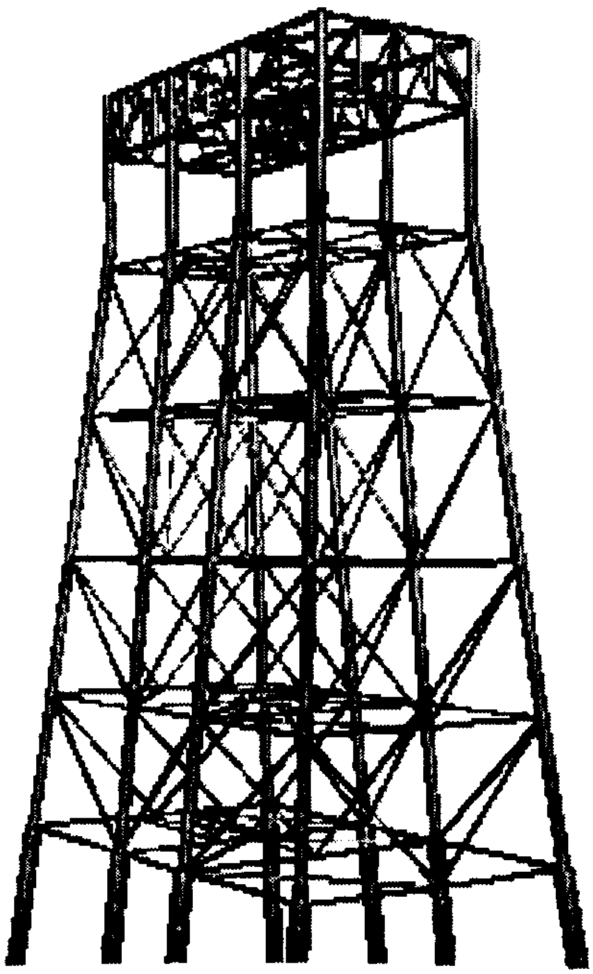
# USFOS Model

- Only major structural members modeled
- Grouted pile/leg member used leg diameter (39 in.) and double the leg thickness (1.0 in.)
- Initial imperfection based on Chen's buckling curve for critical braces
- PMB PAR program developed non-linear springs, but T-Z and Q-Z modeled as equivalent linear springs
- Rigid joints assumed due to grout

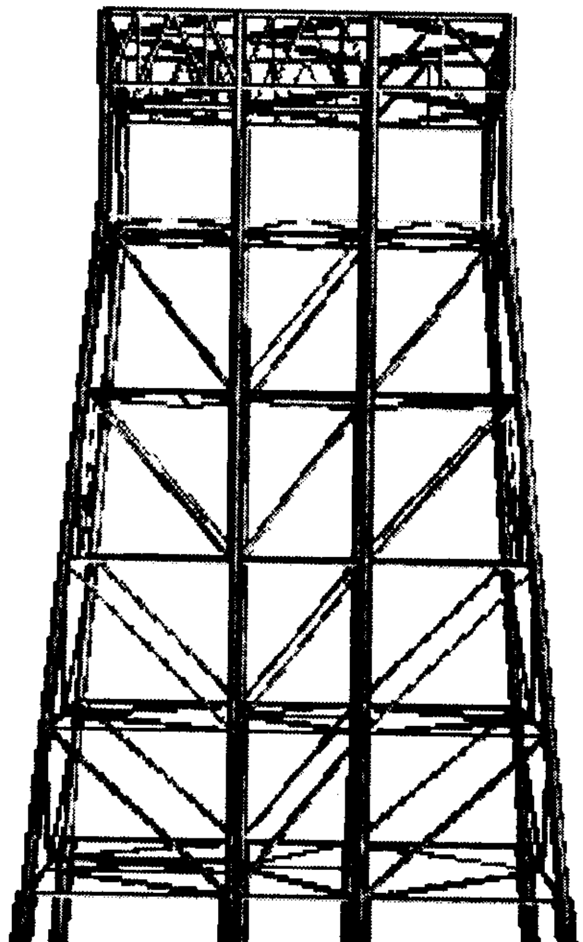
# Orientation



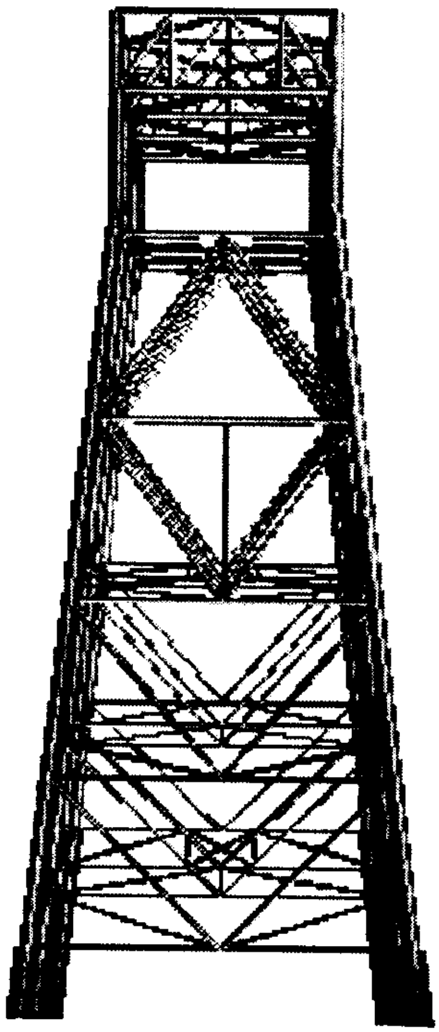
# Isometric



# Broadside Elevation



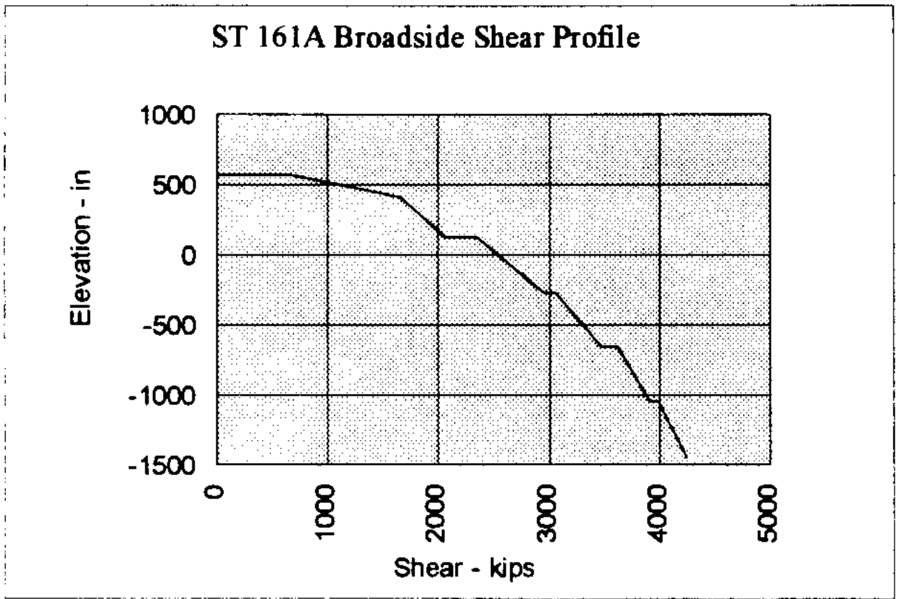
# End-on Elevation

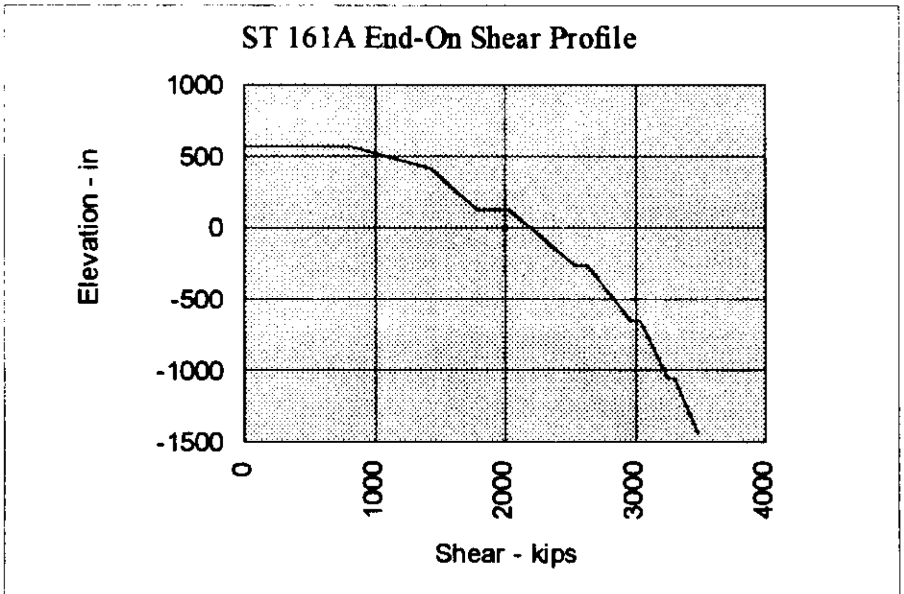


# Loading Information

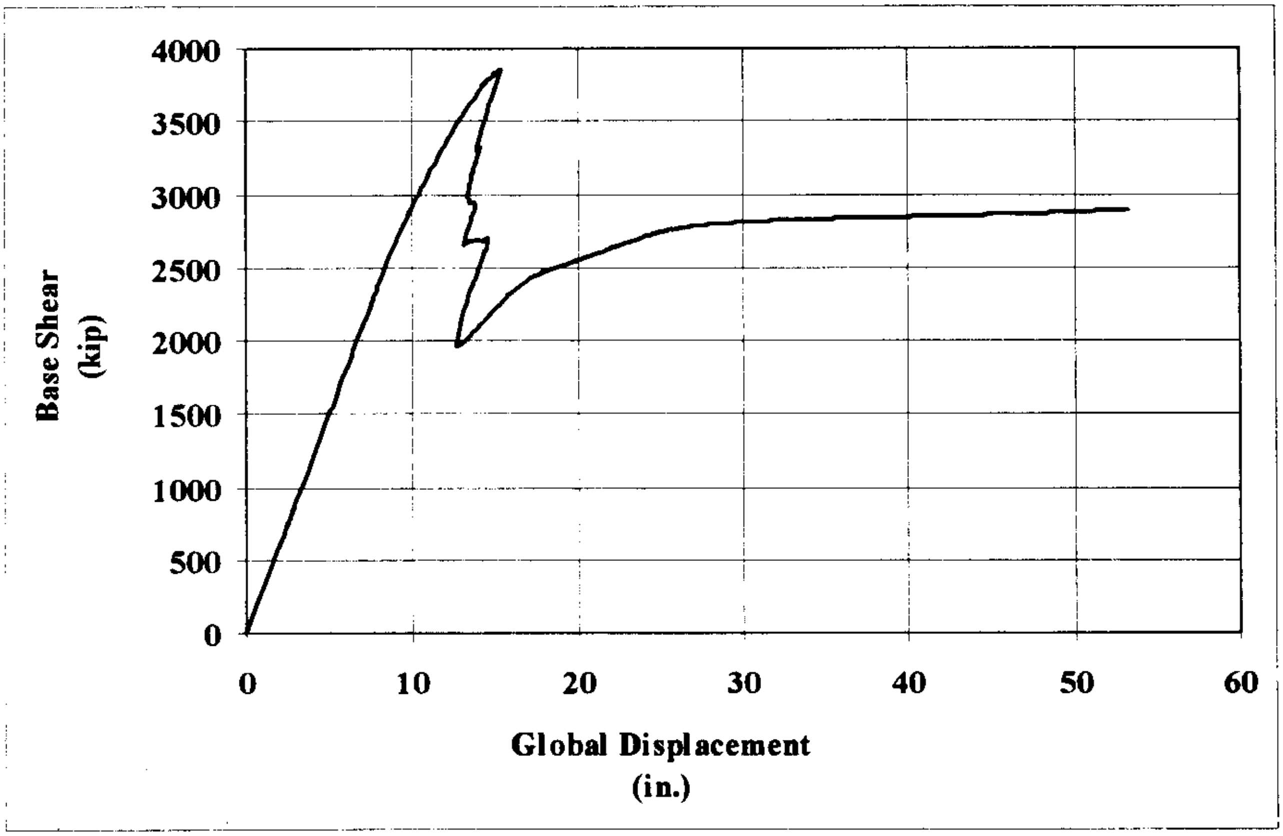
- Assumed marine growth = 1.5 in.
- Cd = 1.2
- Cm = 1.2
- wkf = 0.88
- Broadside loading
  - H = 64 ft, T = 13.3 sec.
  - In-line current = 31 in/sec, cbf = 0.80
- End-on loading
  - H = 72 ft, T = 14.6 sec
  - In-line current = 2.6 in/sec, cbf = 0.70

# Loading Profiles



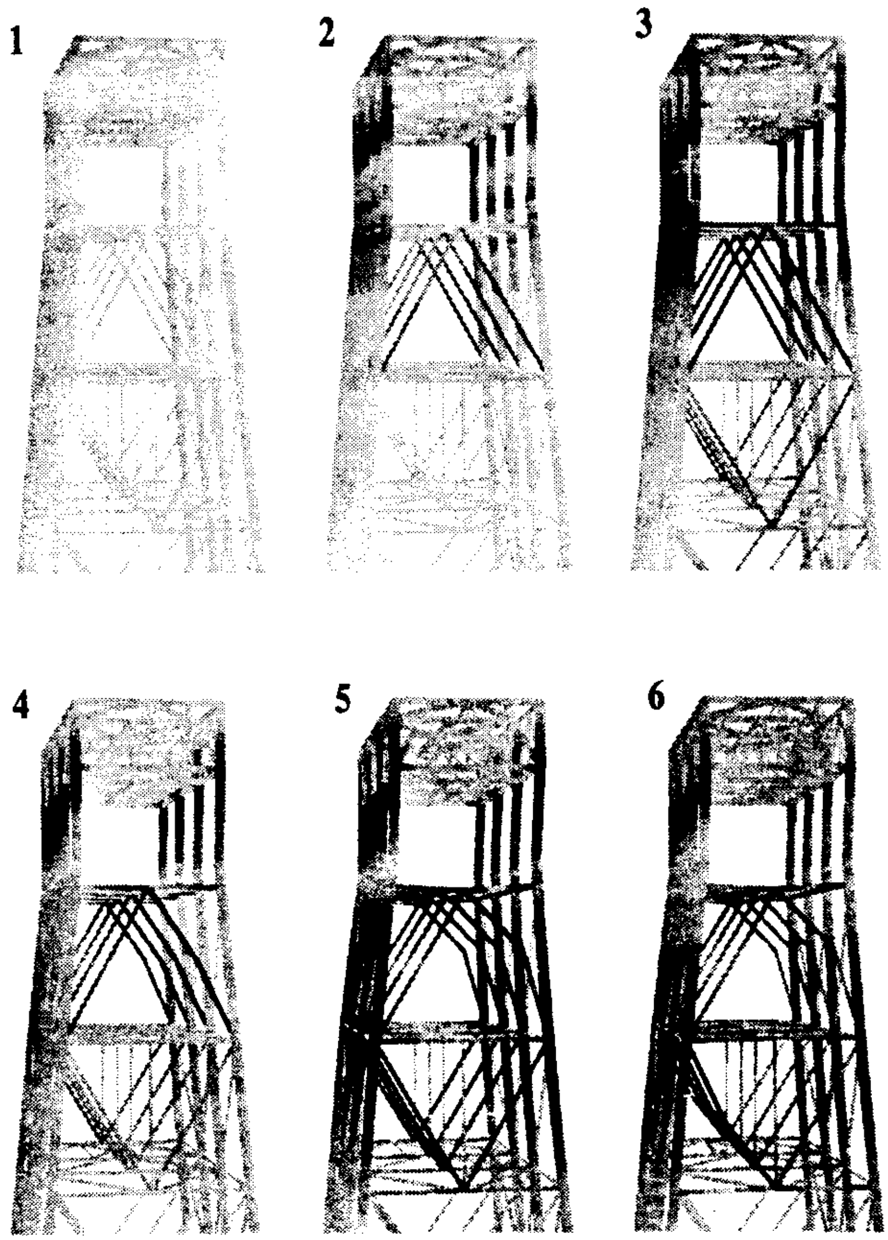



# Broadside Force-Displacement History

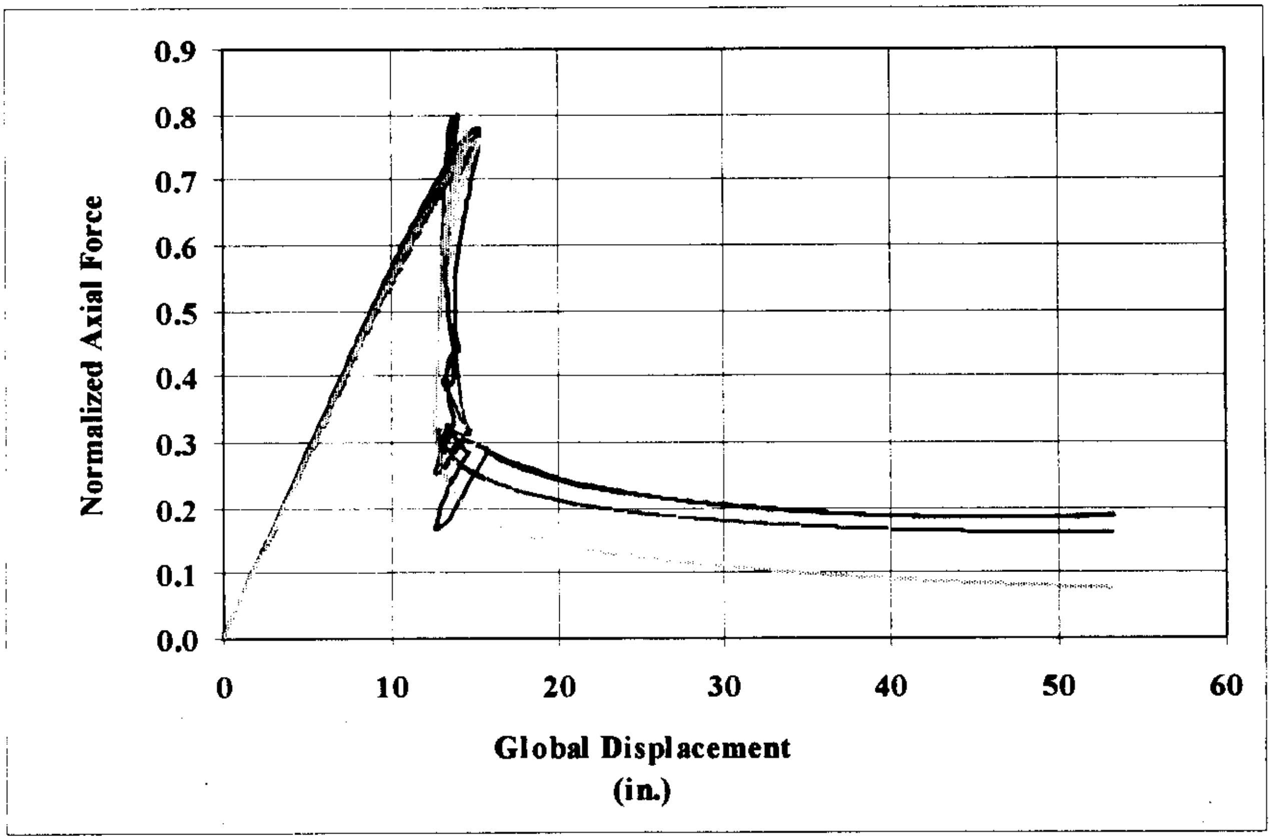



## Maximum base shear = 3,861 kips

# Broadside Failure Progression

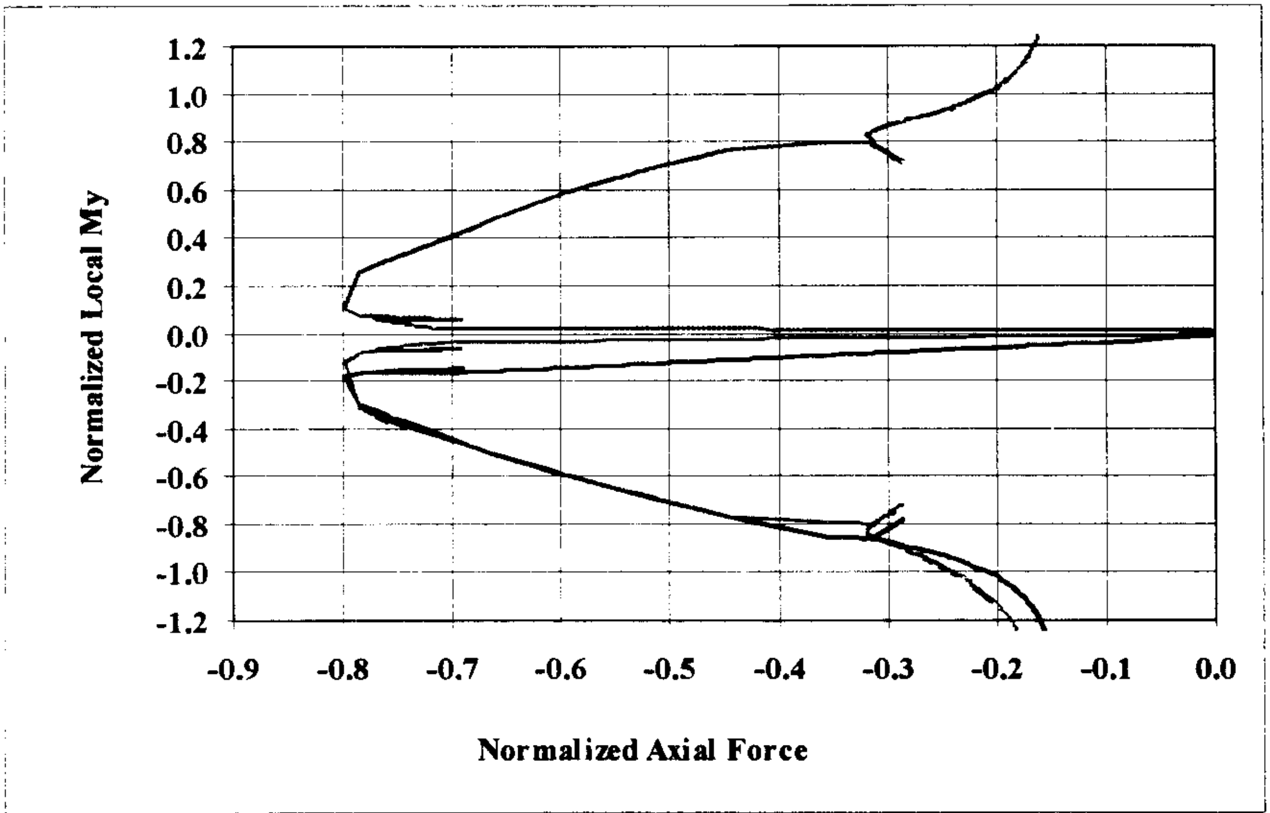


# Broadside Critical Brace Axial Force History



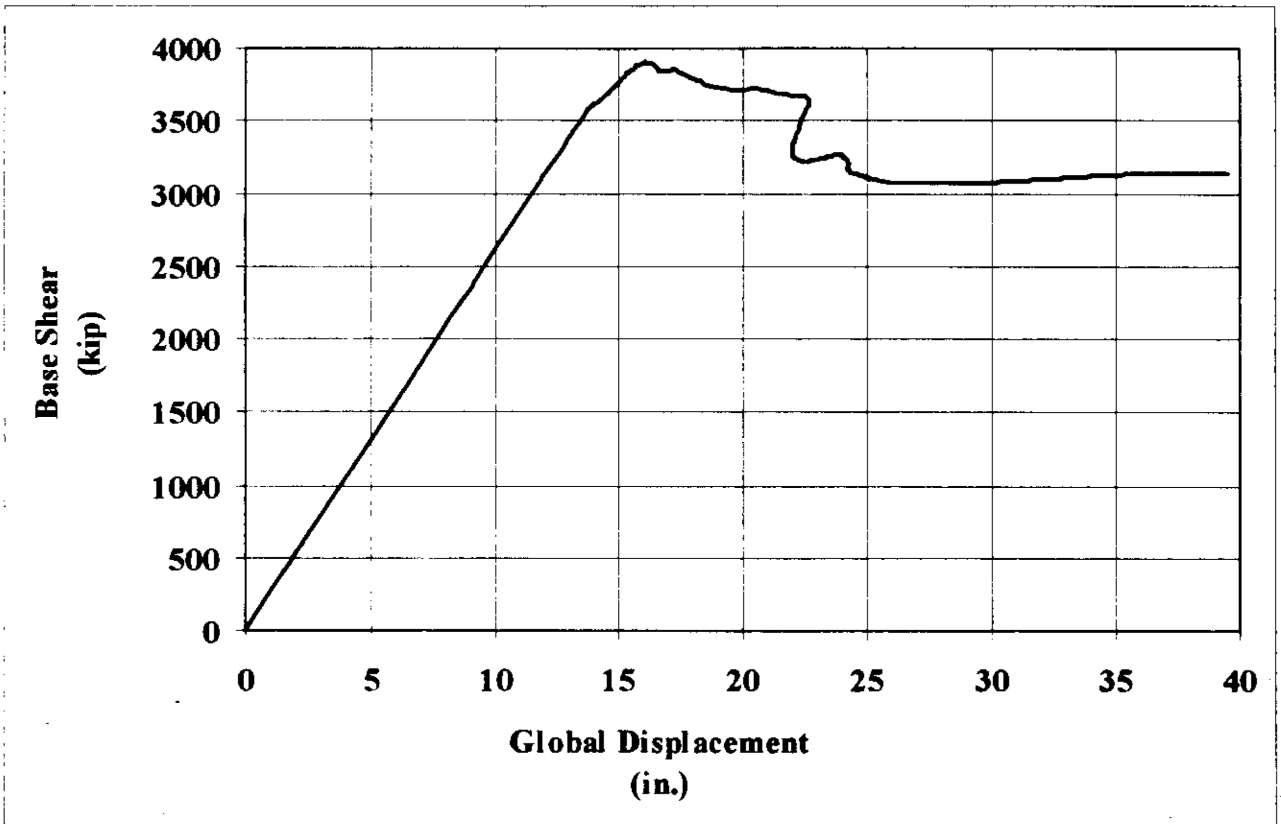

## Values normalized by plastic capacity

# Broadside Critical Brace P-M Interaction



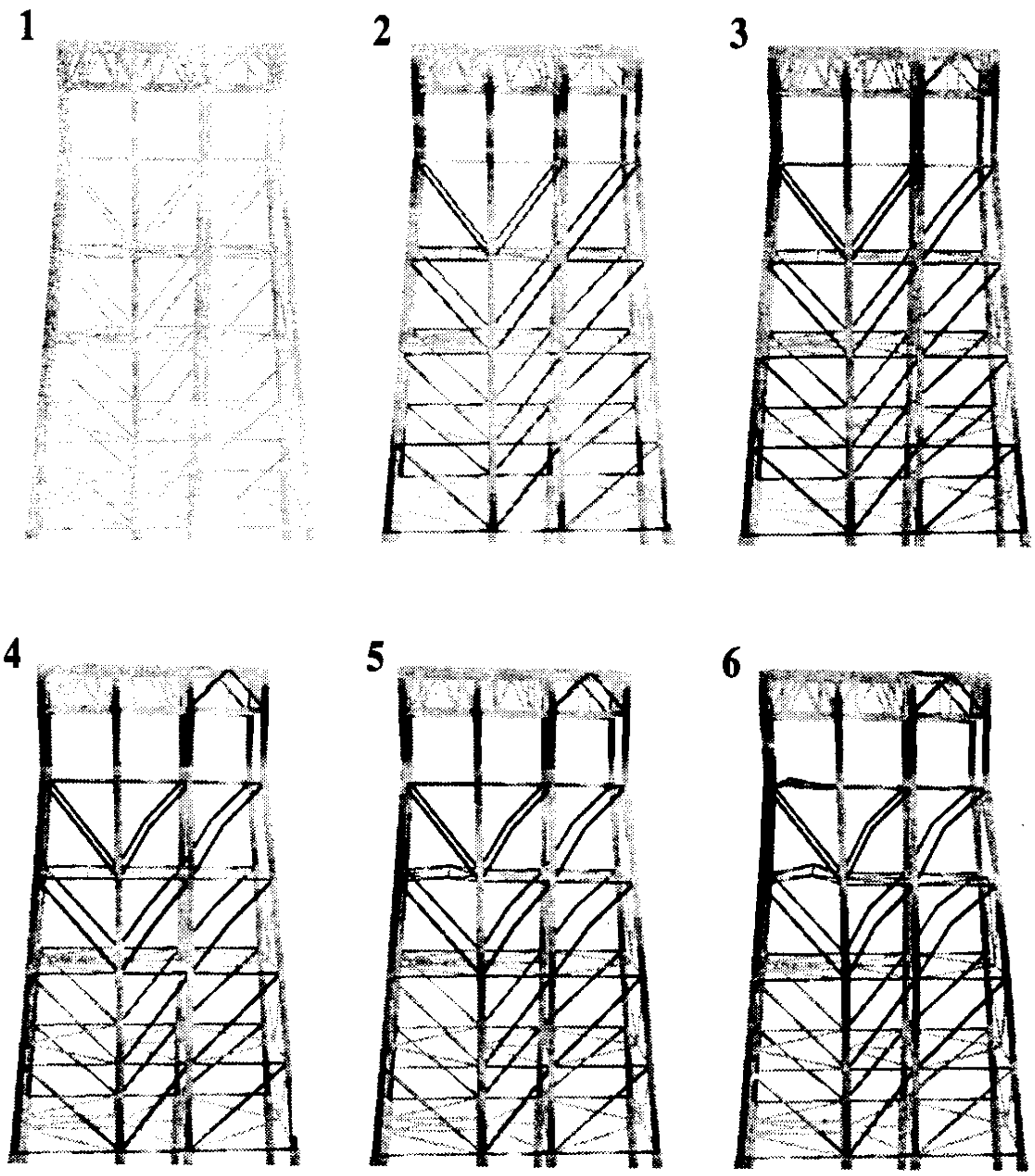

## Values normalized by plastic capacity

# End-on Force-Displacement History

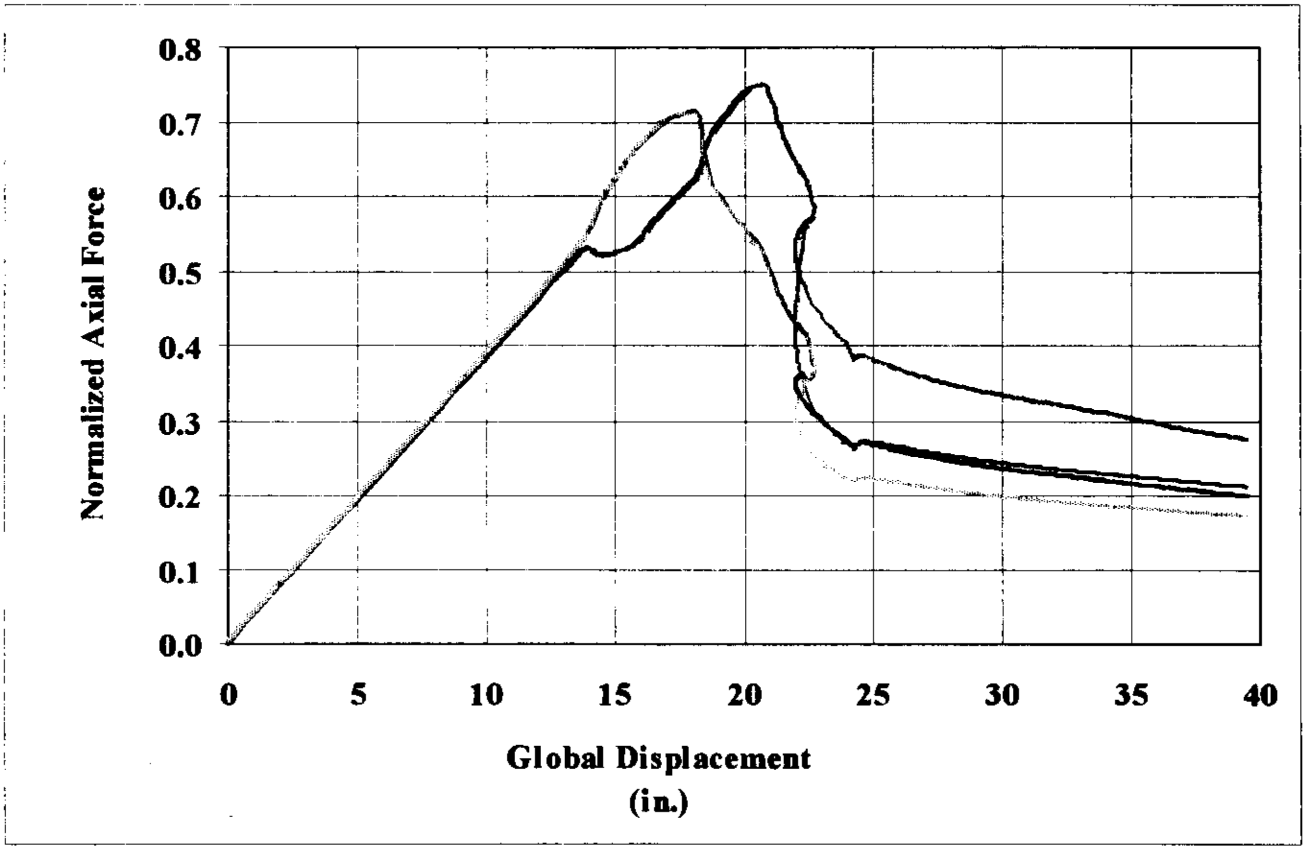



## Maximum base shear = 3,905 kips

# End-on Failure Progression

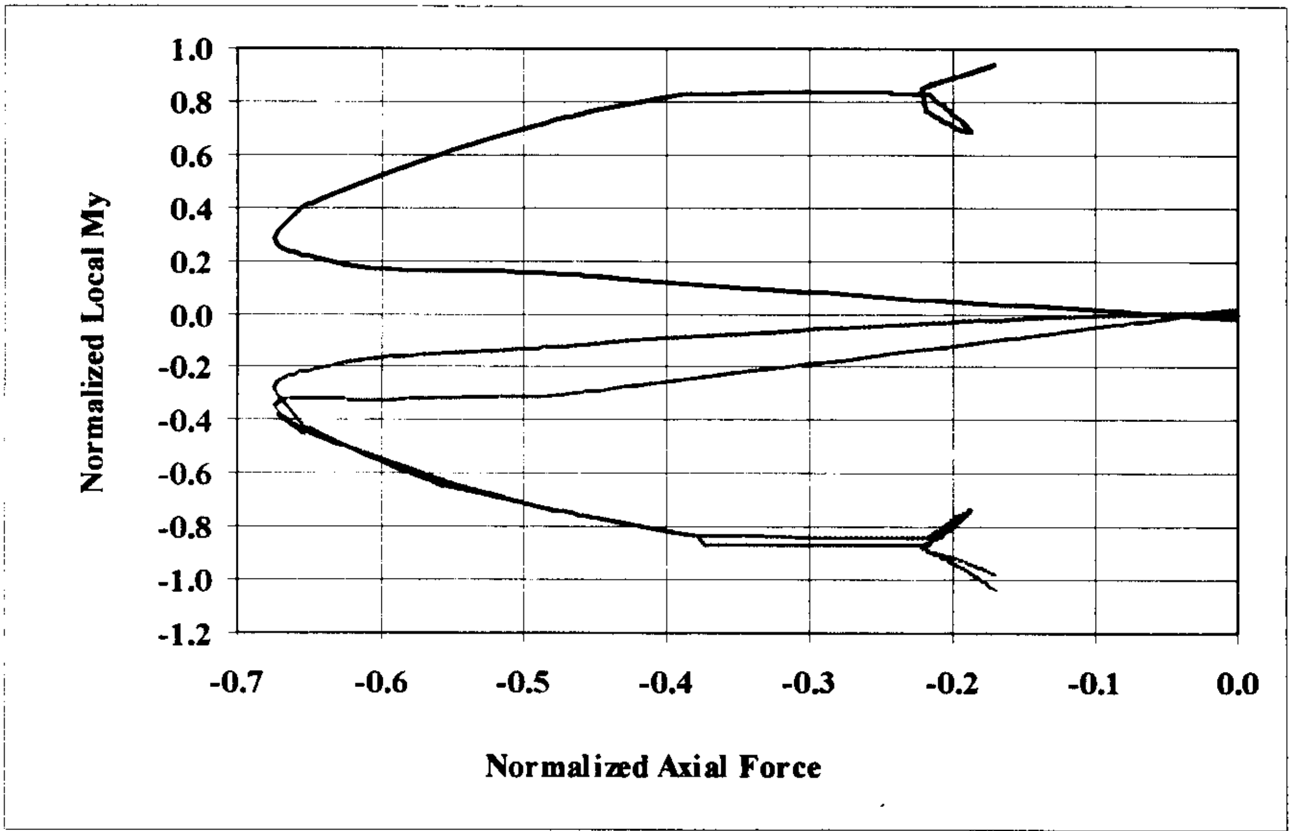


# End-on Critical Brace Axial Force History




## Values normalized by plastic capacity

# End-on Critical Brace P-M Interaction




## Values normalized by plastic capacity

# Comparison with Actual Platform Performance

- ST 161A survived 60-64 ft waves 15° off broadside during Andrew
- USFOS model predicts first member failure at 91% of load from 64 ft broadside wave
- Deck loads are very significant and hence loading is very sensitive to wave height and surge
- Imperfection and member orientation combination is realistic but conservative

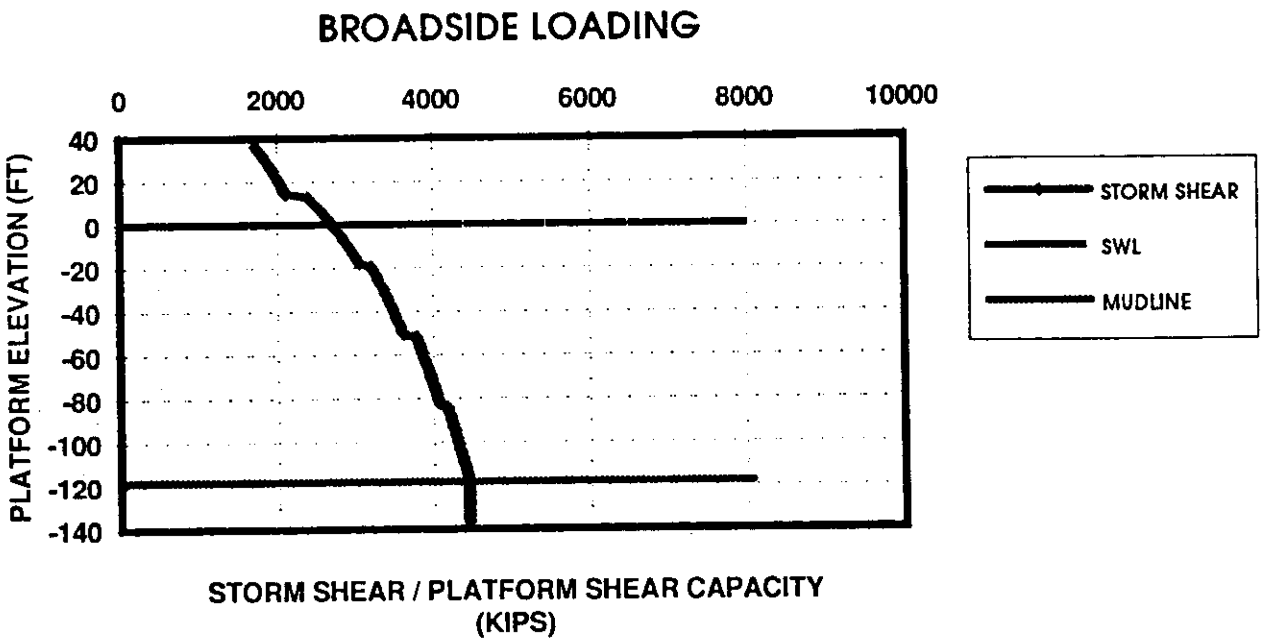
- Conclusion: <u>USFOS model would predict survival during likely Andrew loading</u>

# VERIFICATION CASE STUDIES
## Level 2 Results (AMOCO'S ST161A)

### BROADSIDE LOADING

H = 64 ft; T = 13.3 sec; Uc = 2.6 ft/sec

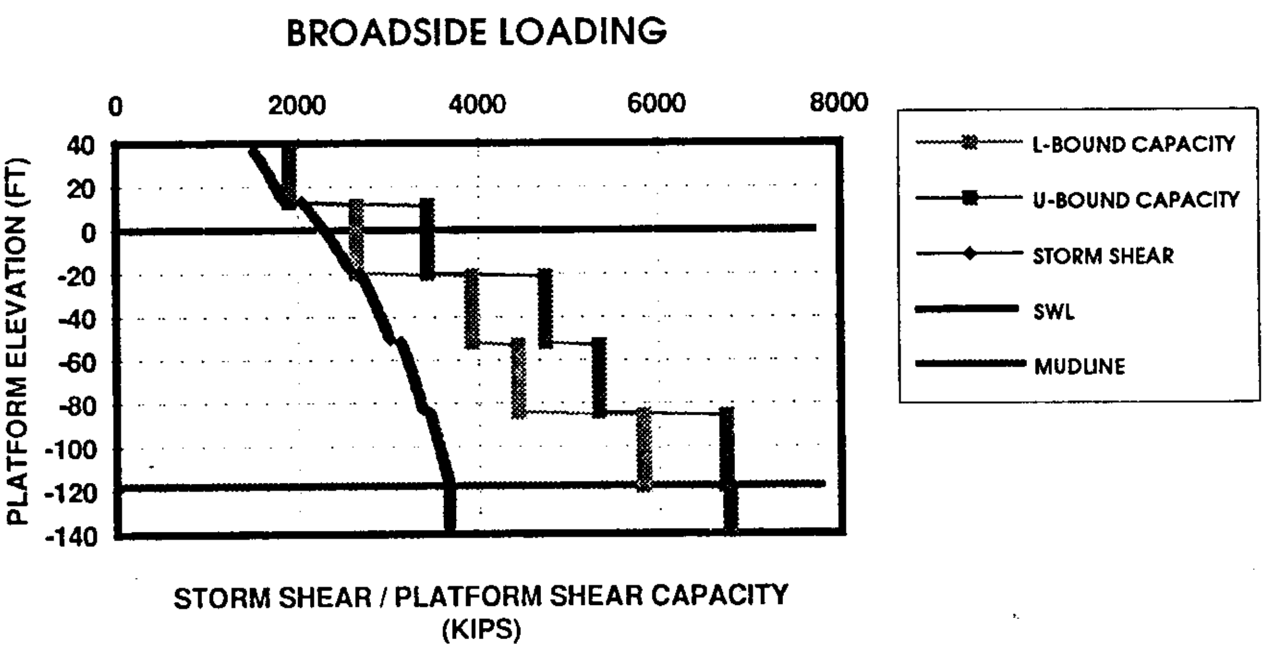
| | Level 4 (SESAM) | Level 2 (ULSLEA) |
|---|---|---|
| Base shear (kips) | 4,252 | 4,428 |
| Jacket Load (kips) | 2,510 | 2,686 |



# VERIFICATION CASE STUDIES
## Level 2 Results (AMOCO'S ST161A)

## BROADSIDE LOADING

| | Level 4 (USFOS) | Level 2 (ULSLEA) |
|---|---|---|
| Base shear at collapse (kips) | 3,861 | 3,670 |

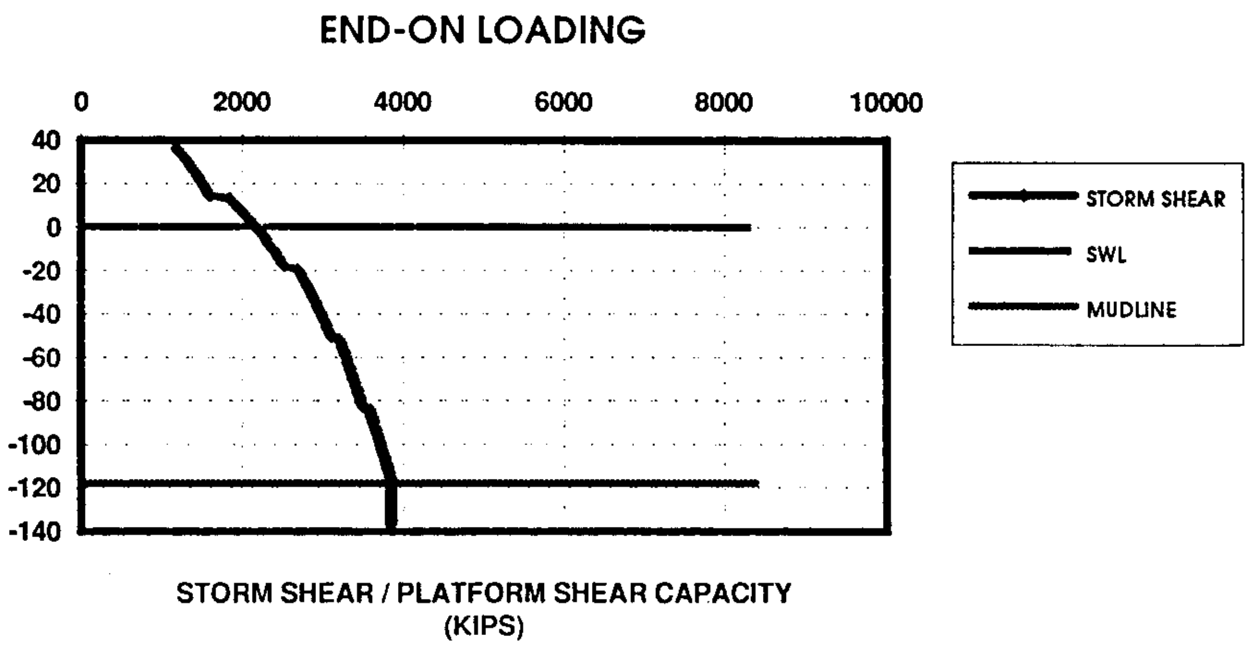


# VERIFICATION CASE STUDIES
## Level 2 Results (AMOCO'S ST161A)

### END-ON LOADING

H = 72 ft; T = 14.6 sec; Uc = 0 ft/sec

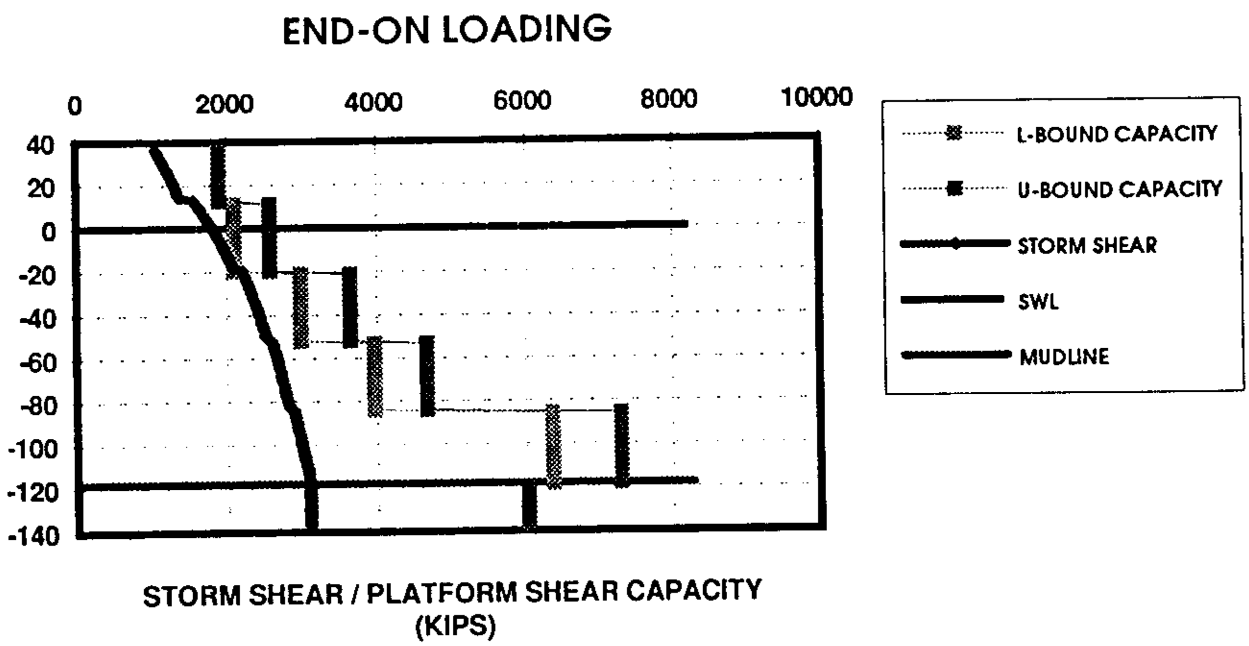
| | Level 4 (SESAM) | Level 2 (ULSLEA) |
|---|---|---|
| Base shear (kips) | 3,487 | 3,814 |
| Jacket Load (kips) | 2,252 | 2,579 |



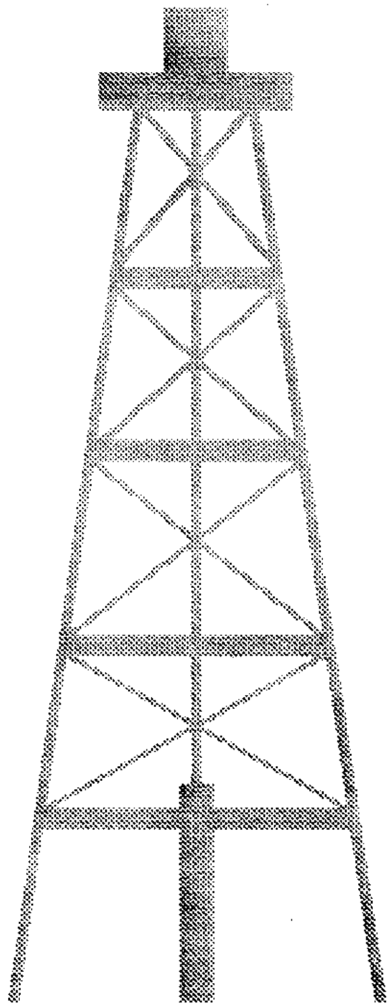
# VERIFICATION CASE STUDIES
## Level 2 Results (AMOCO'S ST161A)

### END-ON LOADING

| | Level 4 (USFOS) | Level 2 (ULSLEA) |
|---|---|---|
| Base shear at collapse (kips) | 3,905 | 3,128 |



# PMB Benchmark Platform



# General Description

- Four leg platform in Ship Shoal area
- Installed in 157 ft of water in 1970
- Broadside and end-on framing battered at 1:11
- Decks located at +33 ft, +43 ft, +56 ft and +71.5 ft
- Three 30 in. and one 48 in. conductors are located in northern half of platform
- Boatlandings on east and south sides

# Platform Details

- Jacket is identical for both primary orthogonal directions
- Legs thickened at joint, 1.25 in. vs. 0.5 in.
- $F_y$ = 43 ksi all members
- 36 in. piles penetrate 355 ft of soft to stiff clay and 28 ft of silty sand (at -197 ft)
- Vertical braces range in size from 16 in. in the seventh (upper) bay to 20 in. in the first bay

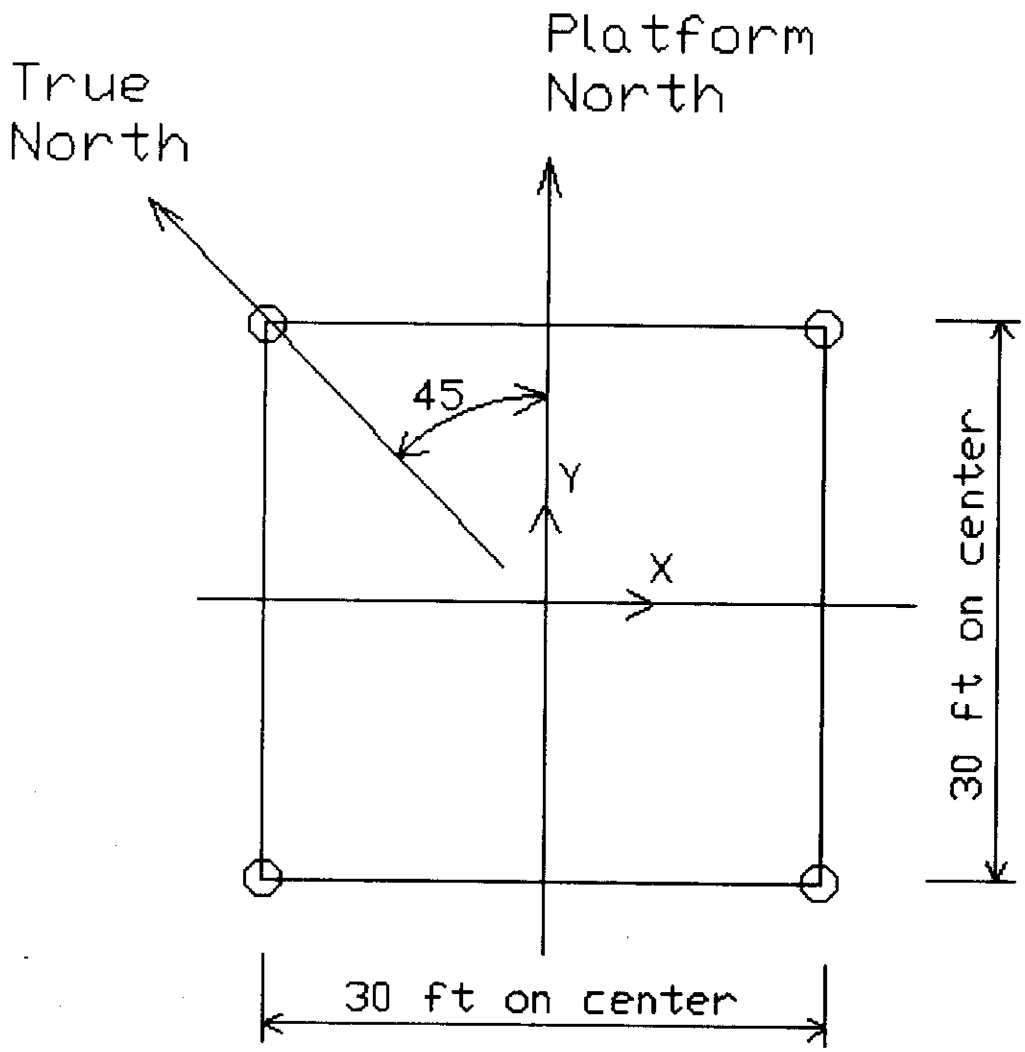
# Level 4 Analysis

- Static pushover analysis
- WAJAC generated hydrodynamic loads
- Rigid and flexible foundation assumptions both analyzed

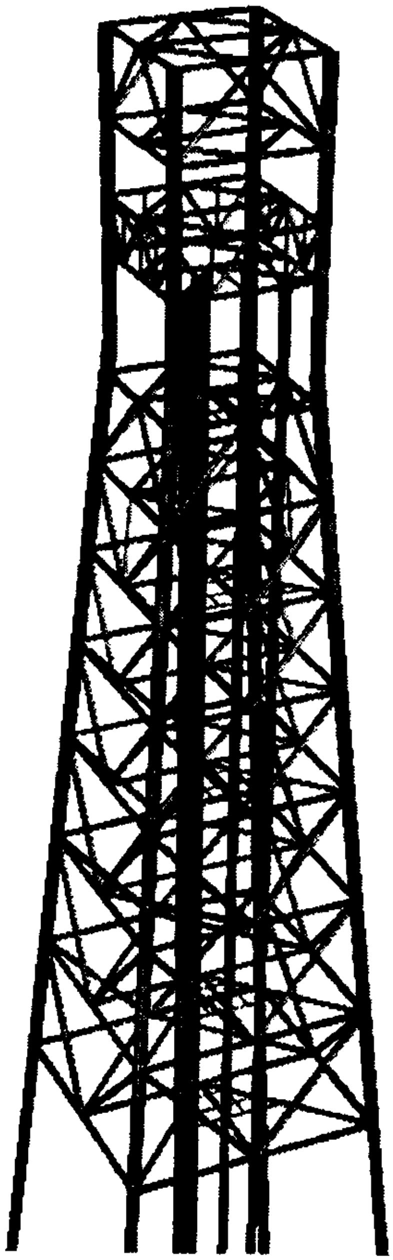
# USFOS Model

- Only major structural members modeled
- Pile/leg annulus ungrouted, thus, jacket joints slaved transversely to pile members. Piles, jacket and deck legs rigidly connected at top
- Initial imperfection based on Chen's buckling curve for critical braces
- Non-linear soil springs developed using API guidelines (static)
- Rigid joints assumed due to thickened sections

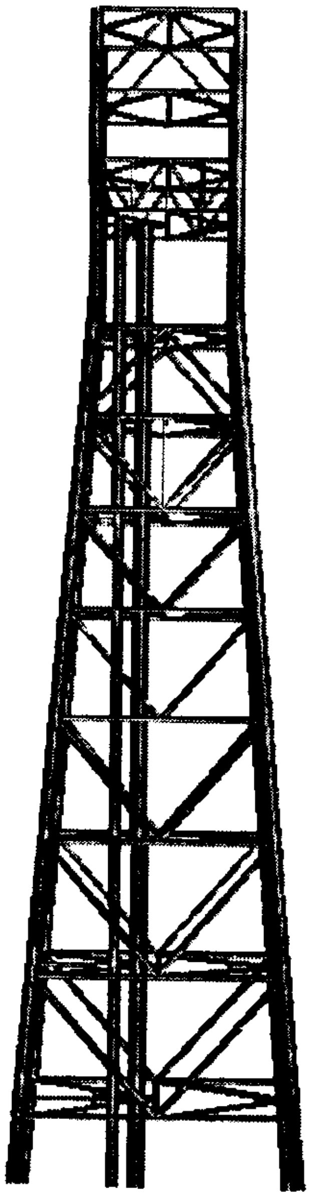
# Orientation



# Isometric



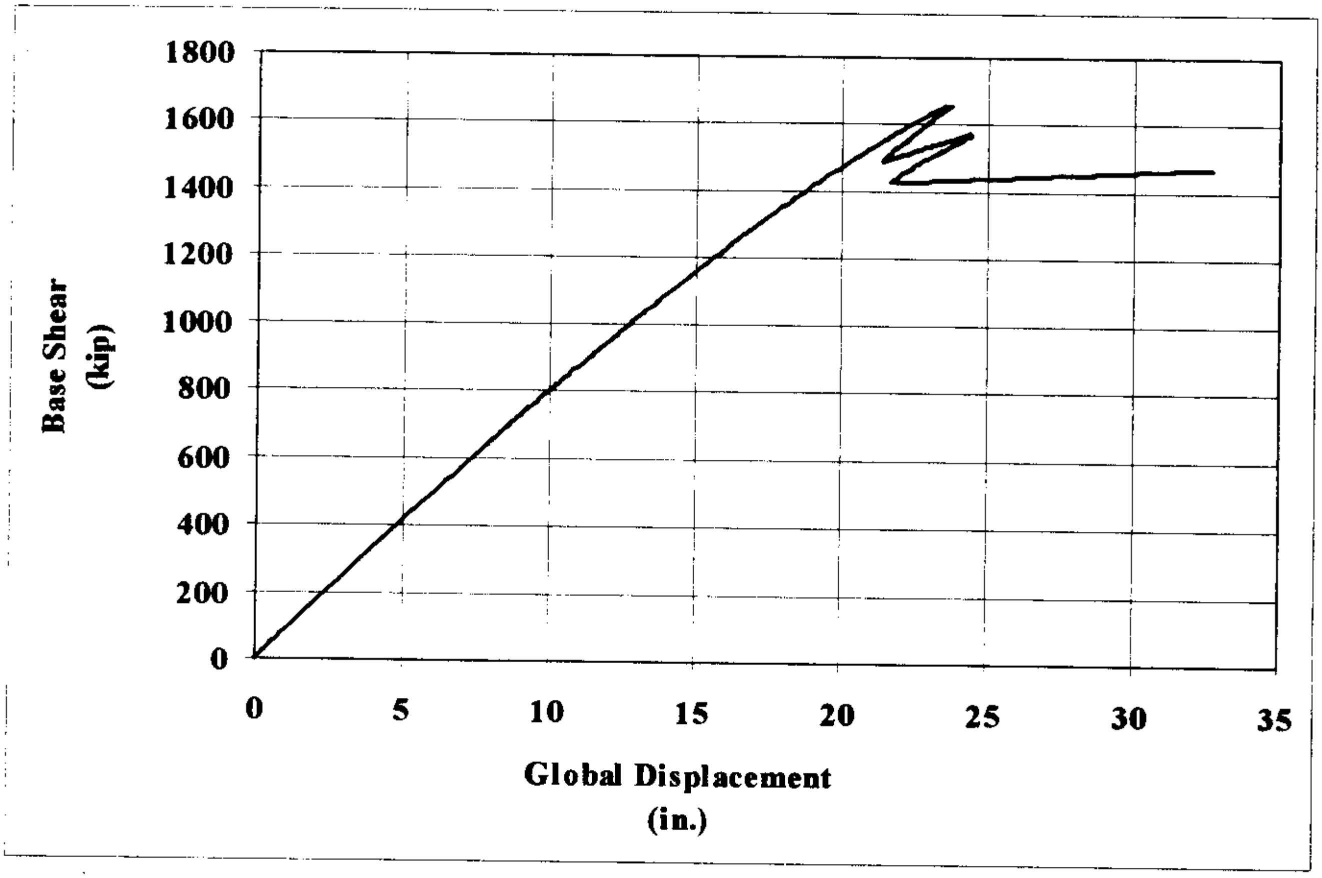
# Side Elevation



# Loading Information

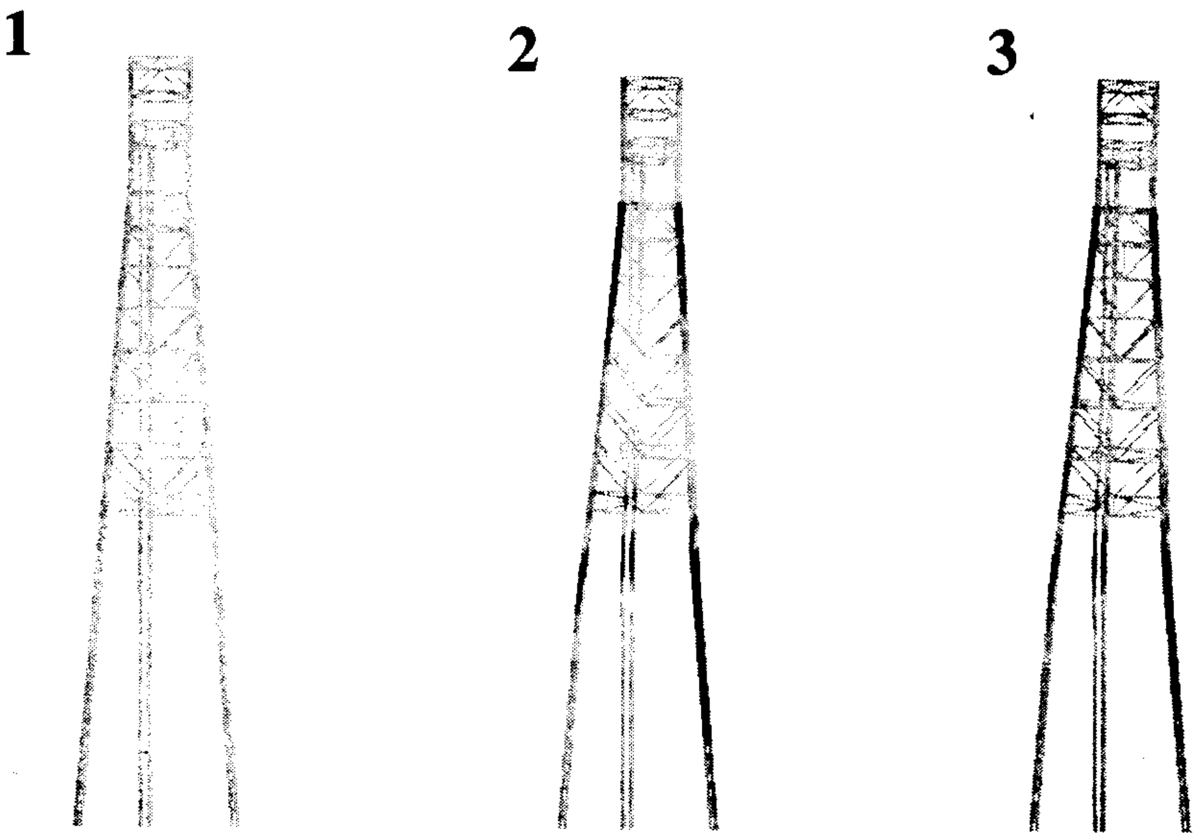
- Assumed marine growth = 1.5 in.
- Cd = 1.2
- Cm = 1.2
- wkf = 0.88
- Broadside loading
  - H = 67 ft, T = 14.3 sec.
  - In-line current = 37 in/sec, cbf = 0.80

# Force-Displacement History

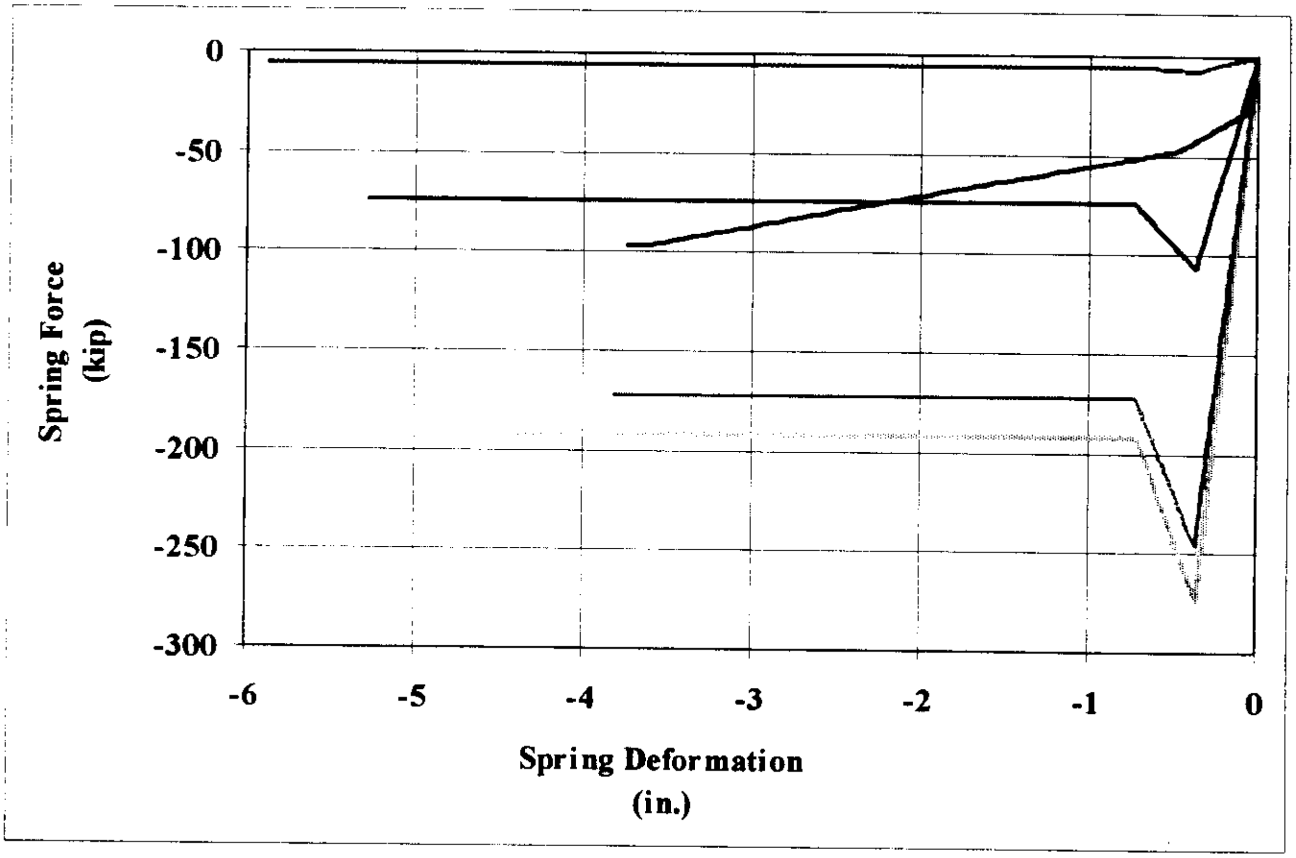



**Maximum base shear =1,673 kips**

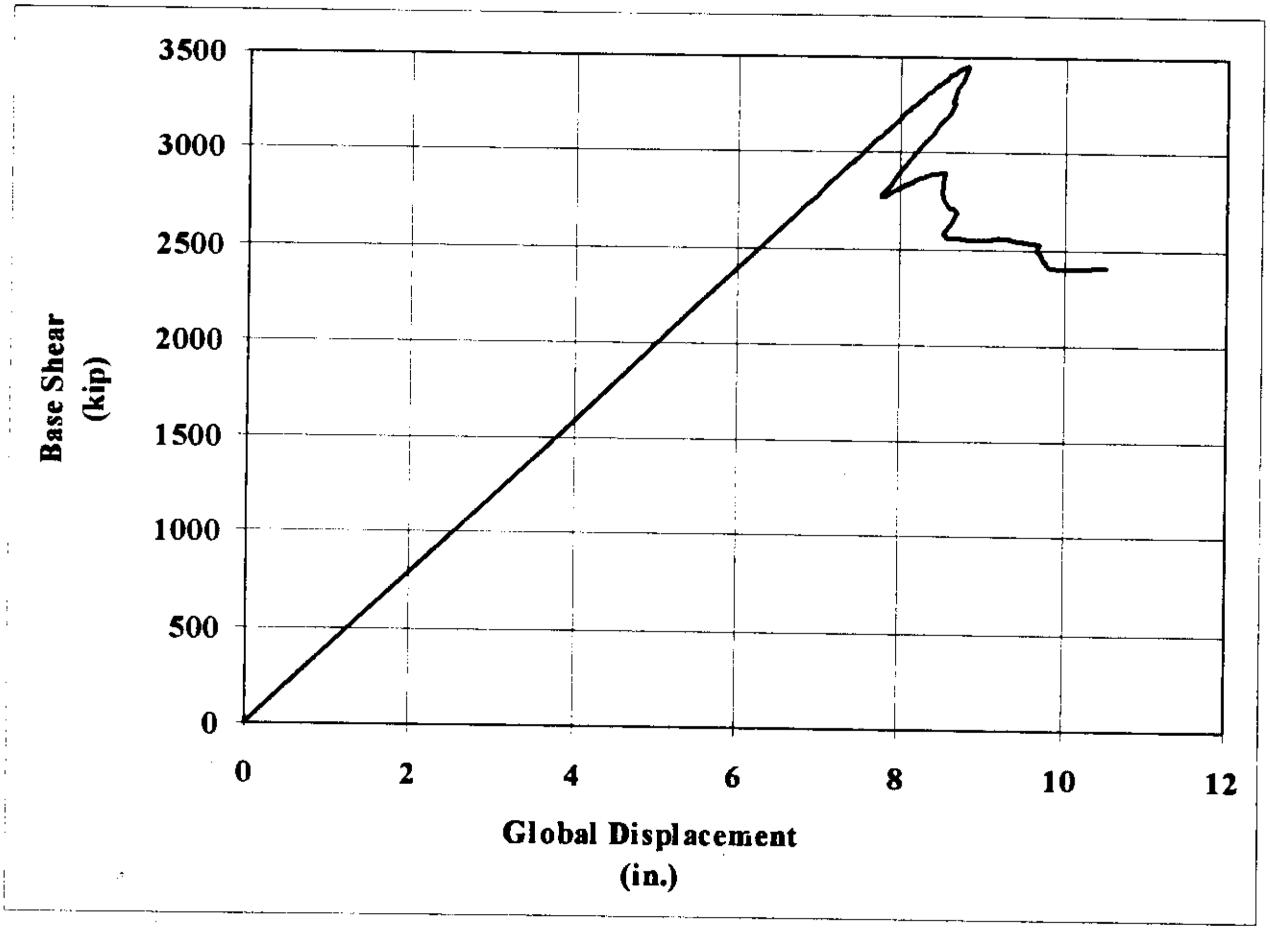
# Failure Progression



# Compression T-Z and Q-Z Soil Spring Force History

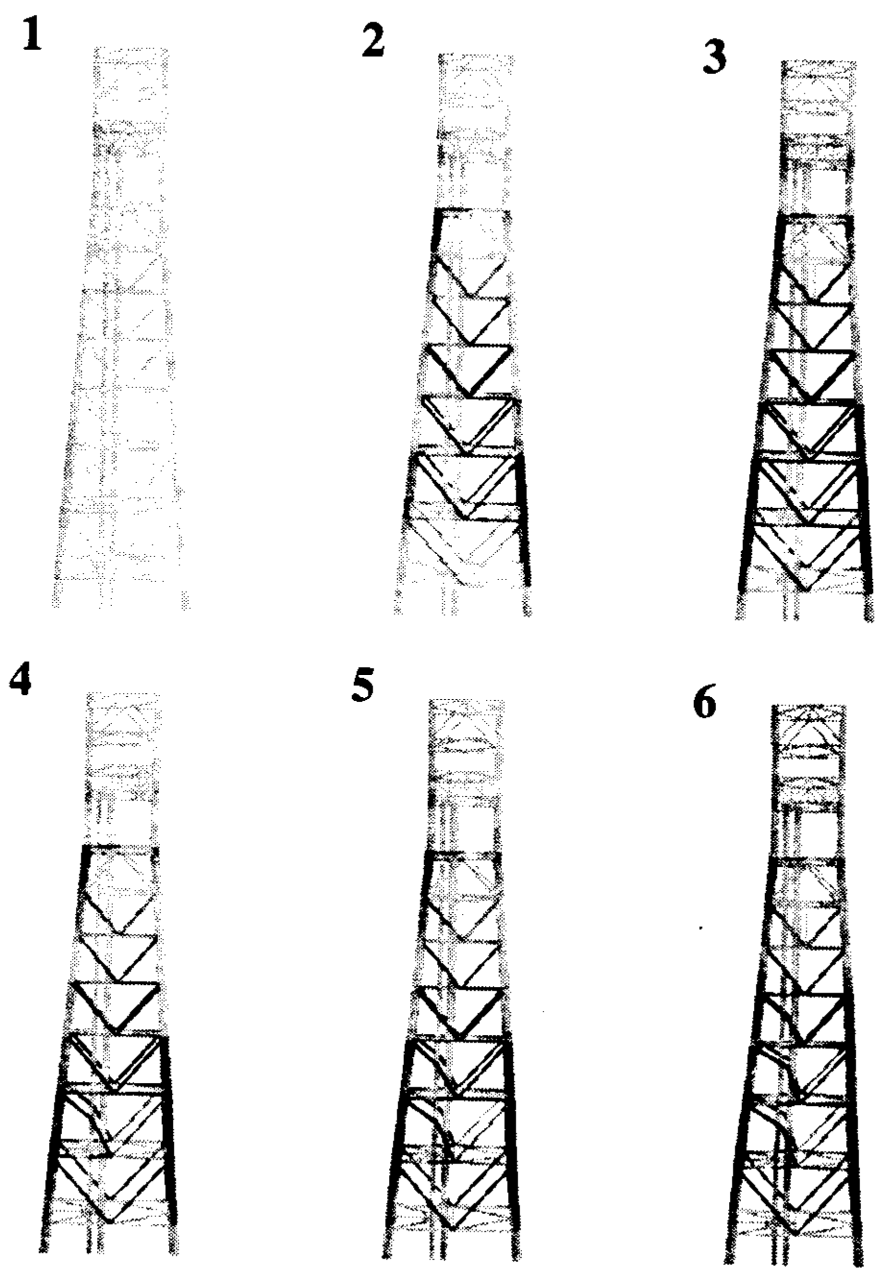


# Fixed Base Force-Displacement History
## (Dynamic Pile Capacity Case)

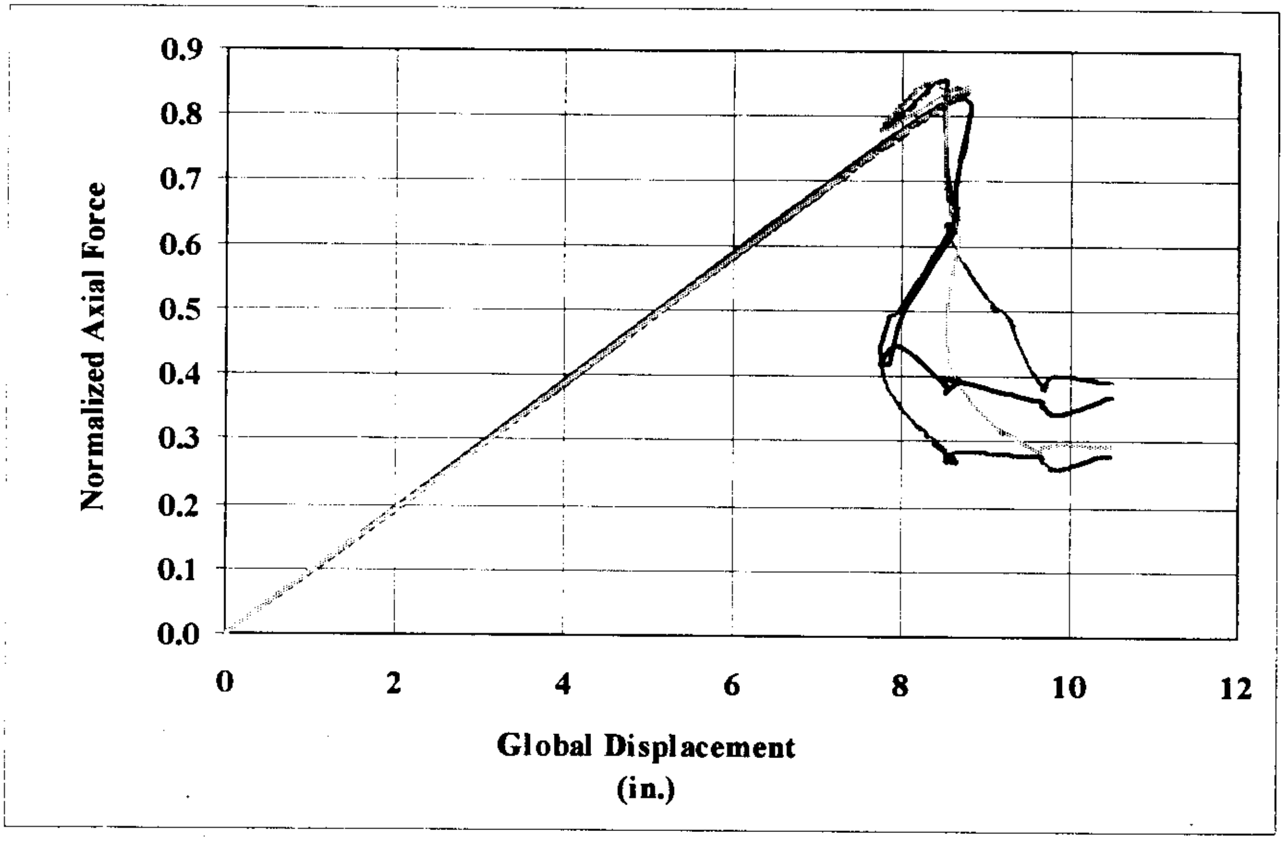



**Maximum base shear = 3,440 kips**

# Fixed Base Failure Progression

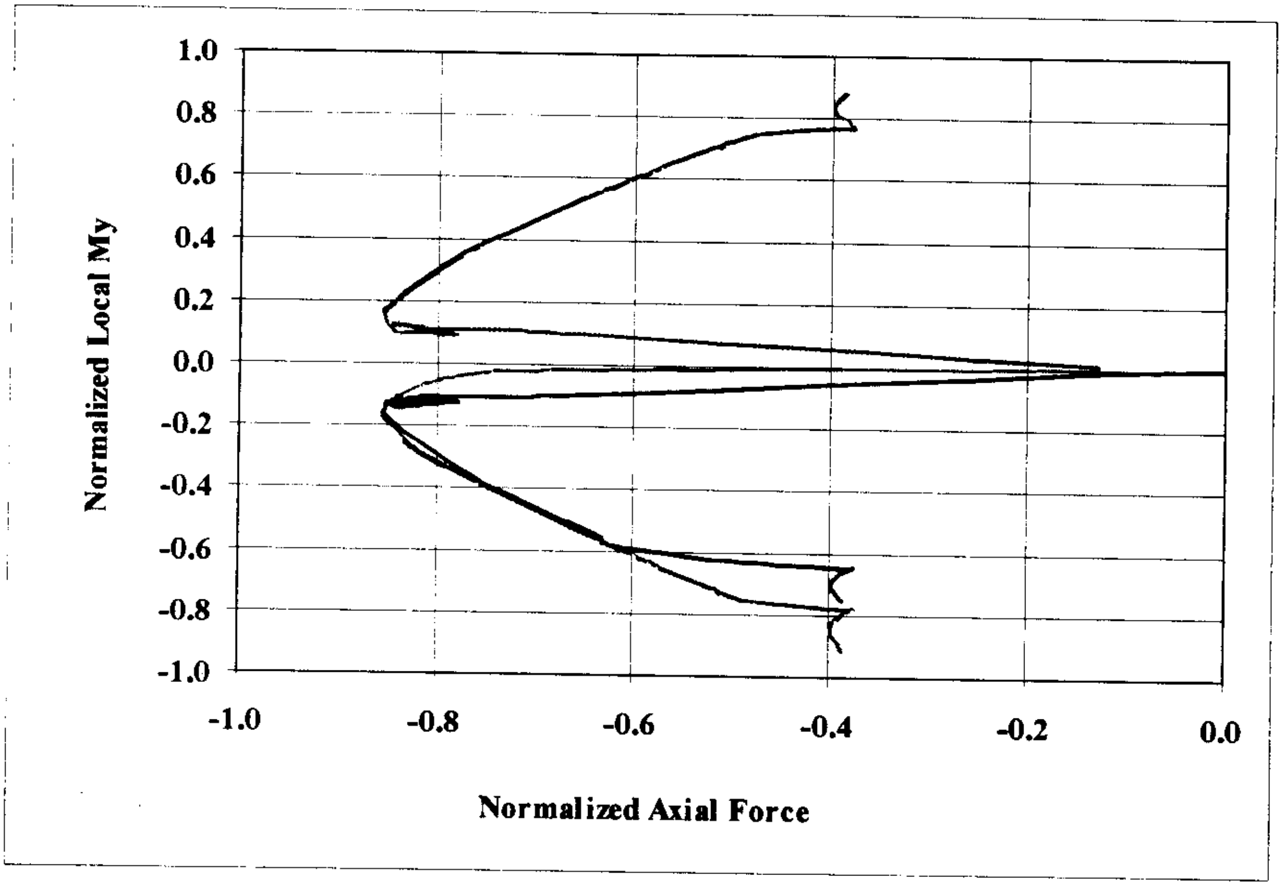


# Fixed Base Critical Brace Axial Force History




## Values normalized by plastic capacity

# Fixed Base Critical Brace P-M Interaction




## Values normalized by plastic capacity

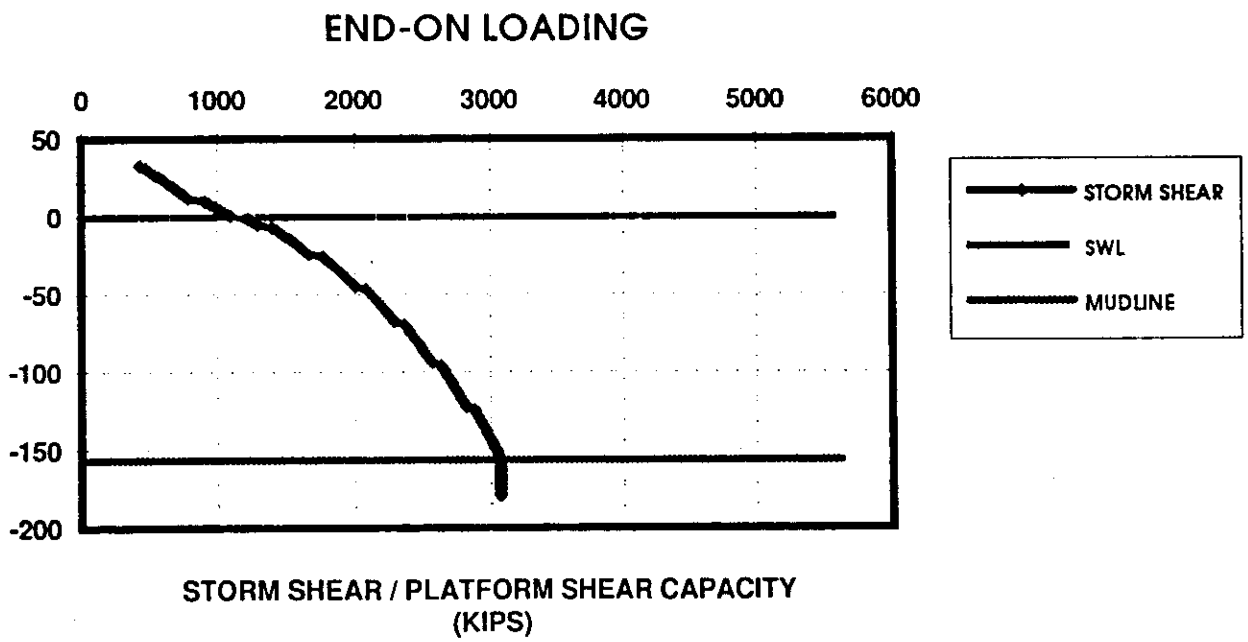
# Research Plans for Next Three Months

- Analyze and document Chevron ST 151H and Chevron ST 151K
- Investigate sensitivity of Level 4 analysis results to input parameters:
  - $F_y$
  - vertical deck forces
  - soil spring assumptions (cyclic, static and dynamic)
- Document benefits and pitfalls of Level 4 analyses based on research experience
- Write final report

# VERIFICATION CASE STUDIES
## Level 2 Results (Benchmark Structure)

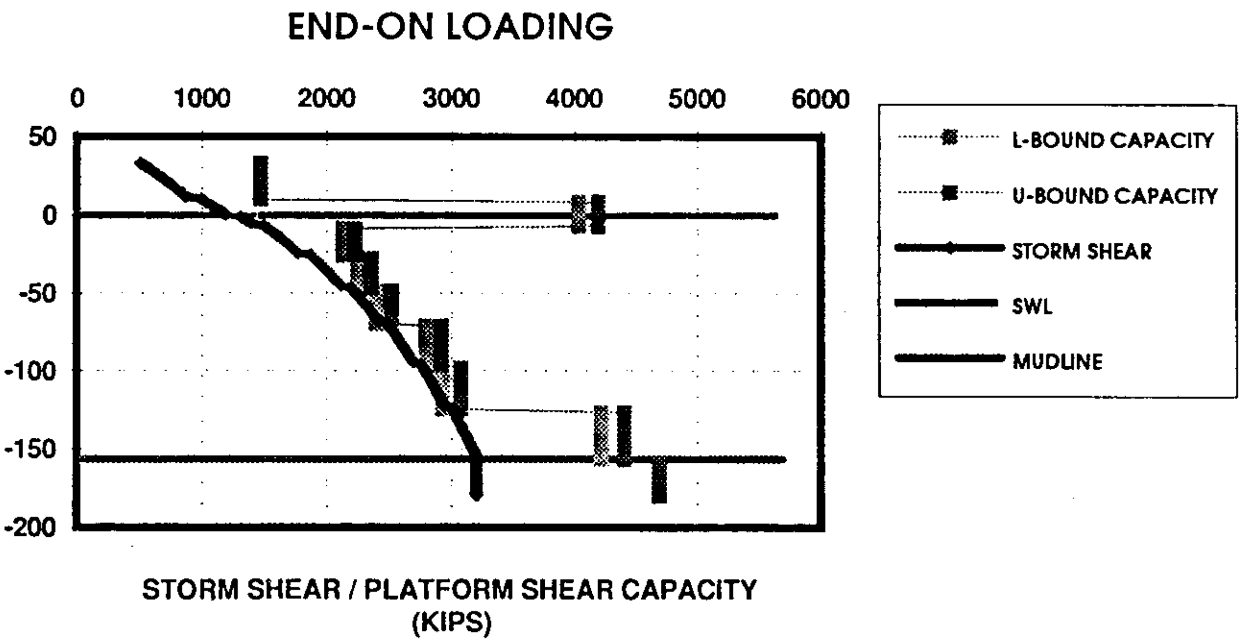
H = 67 ft; T = 14.3 sec; Uc = 3.1 ft/sec

| | Level 4 (SESAM) | Level 2 (ULSLEA) |
|---|---|---|
| Base shear (kips) | 2,656 | 3,055 |
| Jacket Load (kips) | 2,279 | 2,678 |

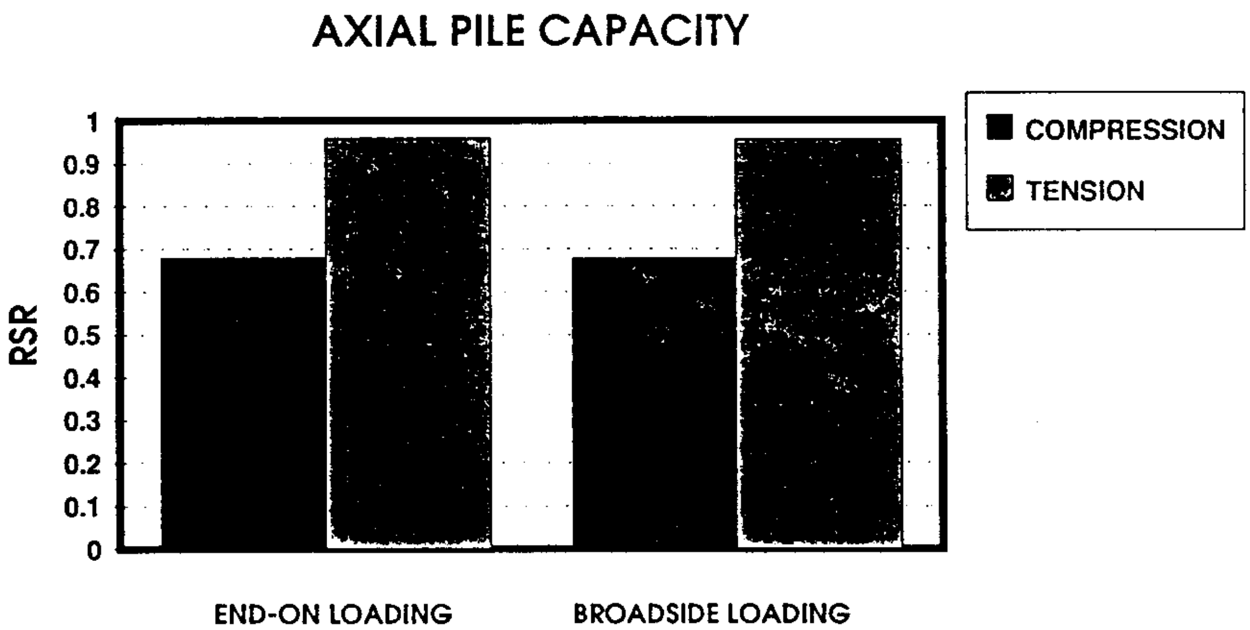


# VERIFICATION CASE STUDIES
## Level 2 Results (Benchmark Structure)

| | Level 4 (USFOS) | Level 2 (ULSLEA) |
|---|---|---|
| Base shear at collapse with fixed base (kips) | 3,440 | 3,212 |



# VERIFICATION CASE STUDIES
## Level 2 Results (Benchmark Structure)

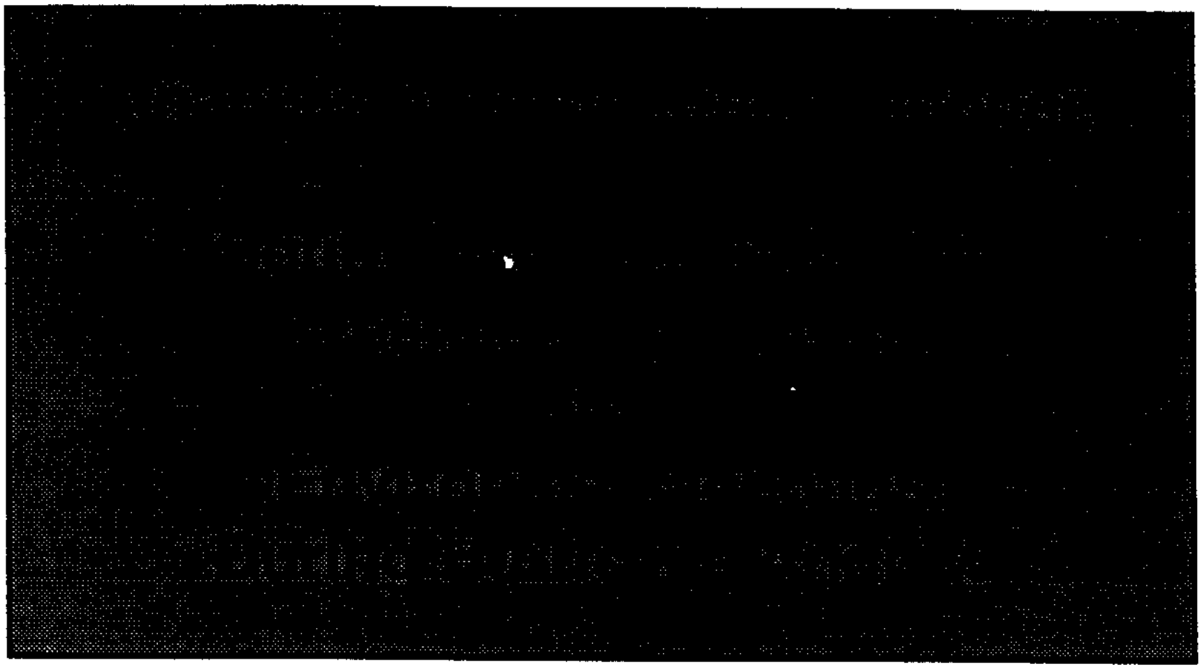


# VERIFICATION CASE STUDIES
## Summary

| Case # | Configuration | Wave Direction | Level 2 Analysis Failure Mode | Level 2 Analysis Base Shear (kips) | Level 4 Analysis Failure Mode | Level 4 Analysis Base Shear (kips) | Ratio USFOS/SCREEN |
|---|---|---|---|---|---|---|---|
| 1 | 8 leg double battered K-braced | End-on<br>Broadside | 1st jacket bay<br>2nd jacket bay | 2,860 (2,700)<br>2,900 | 1st jacket bay<br>2nd jacket bay | 2,607<br>2,935 | 0.91 (0.97)*<br>1.01 |
| 2 | 8 leg double battered K-braced | End-on<br>Broadside | 1st jacket bay<br>1st jacket bay | 3,128<br>3,670 | 1st jacket bay<br>1st jacket bay | 3,905<br>3,861 | 1.25<br>1.05 |
| 3 | 4 leg double battered K-braced | End-on<br>End-on | 4th , 5th and 6th jacket bays<br>Foundation | 3,212<br>1,955 (1,740) | 5th and 6th jacket bays<br>Foundation | 3,440<br>1,673 | 1.07<br>0.86 (0.96)** |

*) Including the shear in jacket legs

**) Including the platform selfweight




*Peter Young*
*Graduate Student Researcher*

*and*

*Professor Robert Bea*

*Department of Civil Engineering*
*University of California at Berkeley*


*7th October 1994*

# OBJECTIVE

Given two roughly identical wellhead protectors subjected to Hurricane Andrew storm shears, determine ability to predict the observed performance of the two structures:

- Wellheads Protector 2 and 3 (WP2/3) collapsed

- Wellhead Protector 4 (WP4) suffered no significant damage.

# THREE APPROACHES TO DETERMINE RESPONSE

- Linear Elastic Analysis Using StruCAD*3D
  - Piles fixed ≈ 5 pile diameters below mudline
  - Approximate linear springs at mudline
  - Nonlinear soil-structure interaction along length of the piles

- Ultimate Limit State Limit Equilibrium Analysis (ULSLEA)

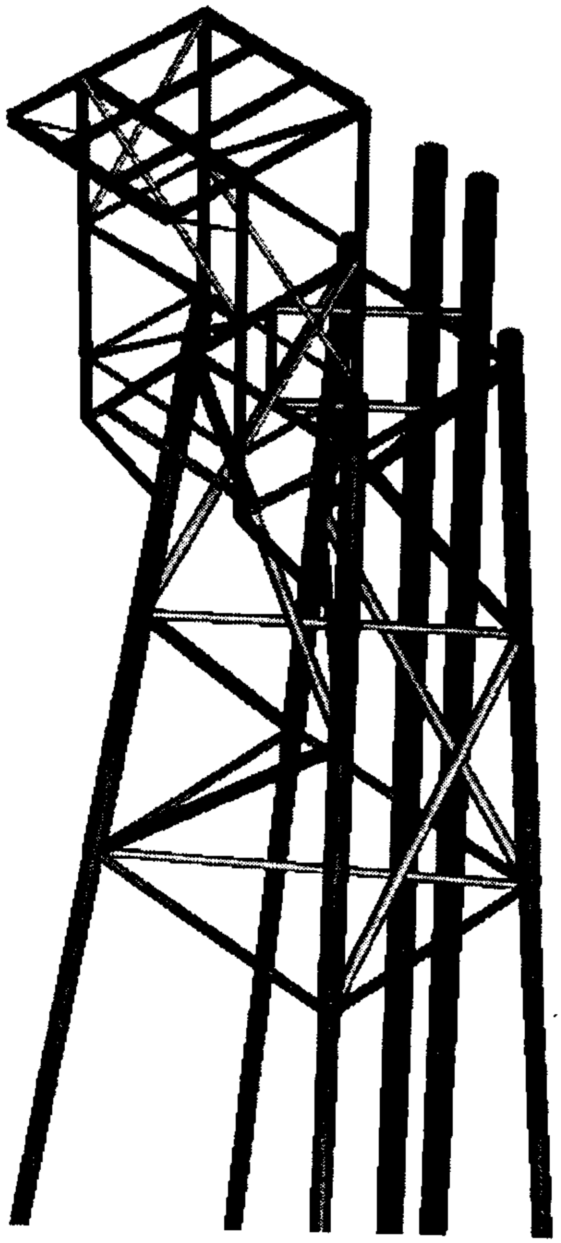
- Nonlinear Static Pushover Analysis Using Usfos
  - Piles pinned at mudline
  - Nonlinear Winkler soil springs

# STRUCTURAL CHARACTERISTICS

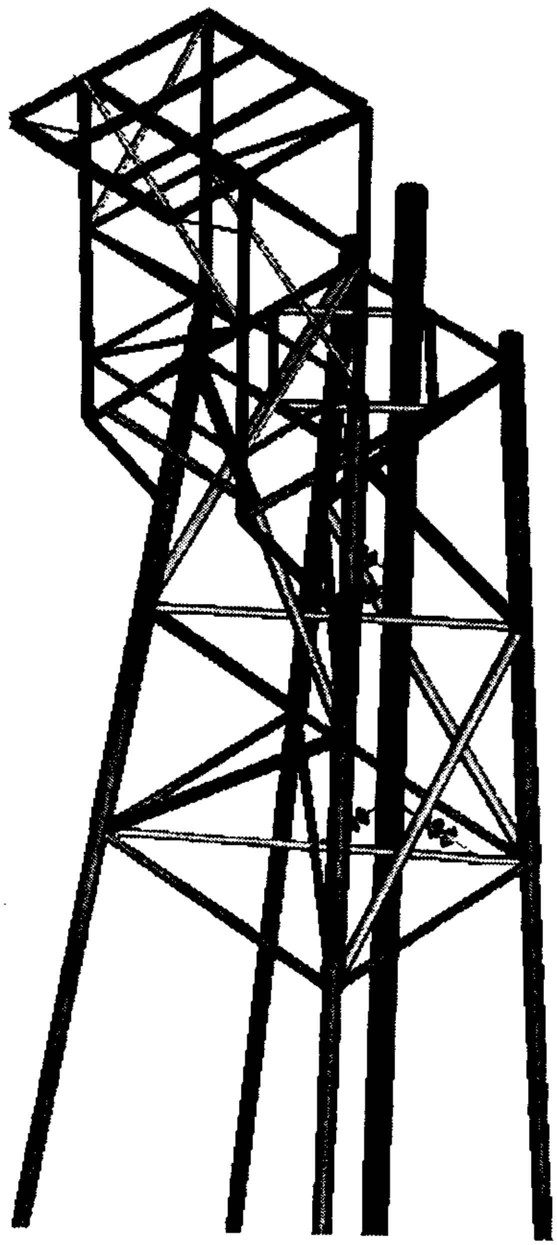
- Wellheads Protector 2 and 3 (WP2/3)
  - 52' mean water depth (MWL)
  - Oriented -45° from true north
  - 2 exterior well caissons (36"ø)

- Wellhead Protector 4 (WP4)
  - 49' MWL
  - Oriented parallel to true north
  - 1 interior well caisson (36"ø)

# 3D ISOMETRIC OF WP2/3



# 3D ISOMETRIC OF WP4



# SOIL CHARACTERISTICS

- Upper Layer of Weak Clays (to depth of 64')

  Shear Strengths = 0.31-0.50 ksf

- Intermediate Layer of Clays (64' to 172')

  Shear Strengths = 0.5-1.5 ksf

- Underlying Layer of Stiff Sands (below 172')

  Shear Strength = 2.0 ksf

# WIND, WAVE AND CURRENT CHARACTERISTICS

- ABS wind profile with maximum velocity of 98 knots

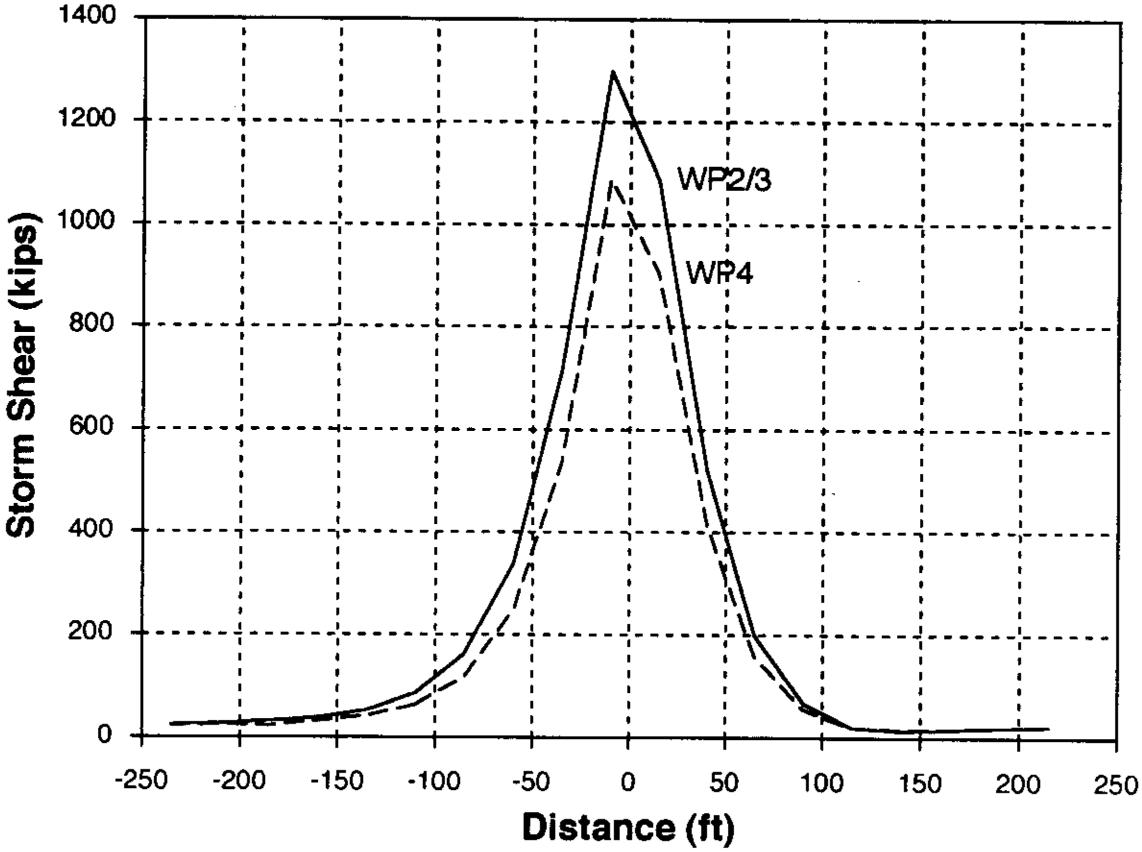
- 40' maximum wave height
  - 9.5 second wave period
  - Stream Function 9th Order wave theory

- Constant 6 ft/sec current over water depth

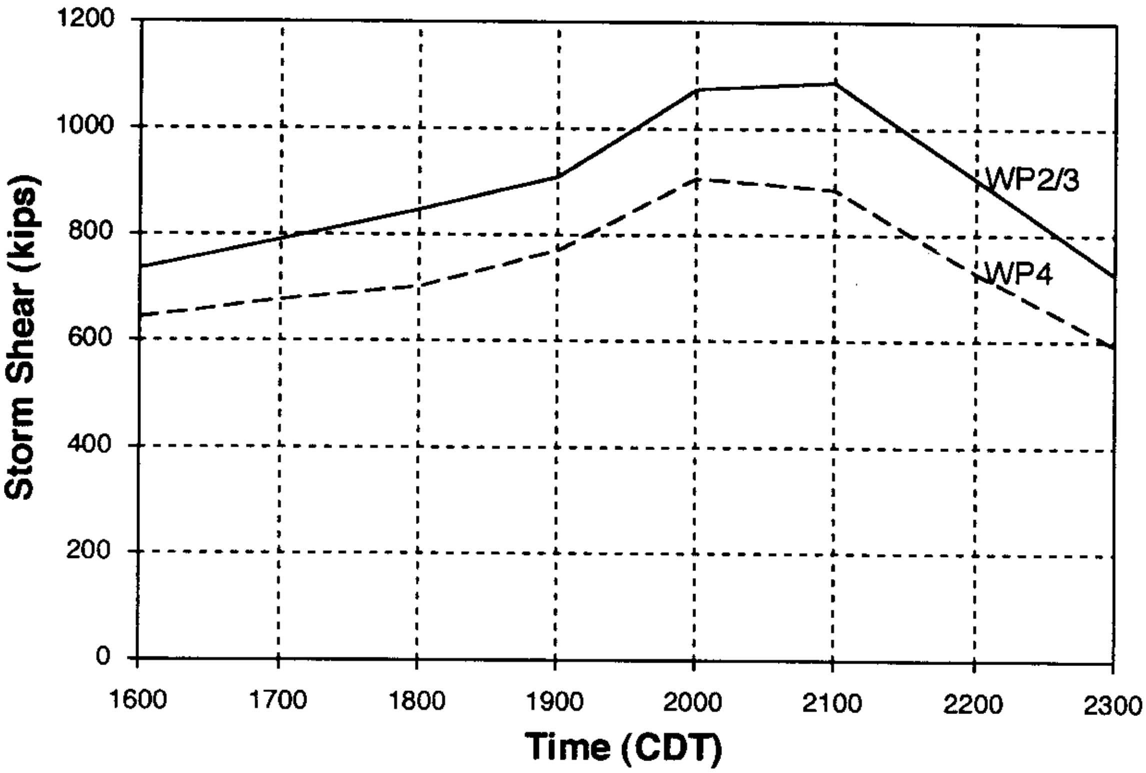
- Cd = 1.2 accounts for marine growth

- Maximum Surge and Tide of 3'

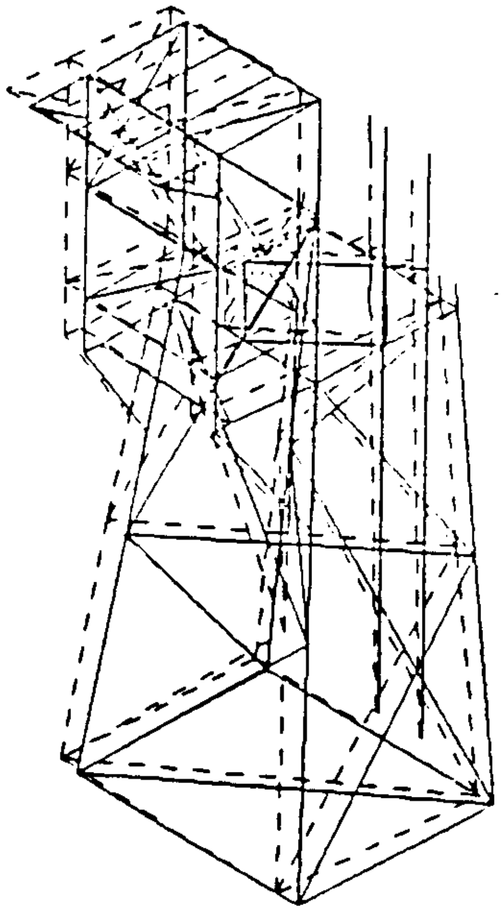
# BS BASE STORM SHEAR VS. DISTANCE TO ORIGIN



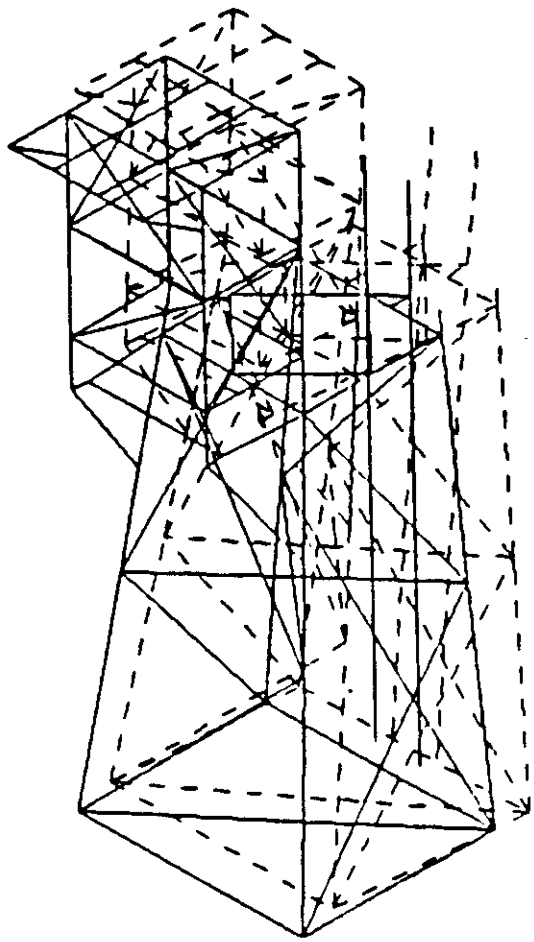
# BASE STORM SHEAR VS. TIME DURING ANDREW



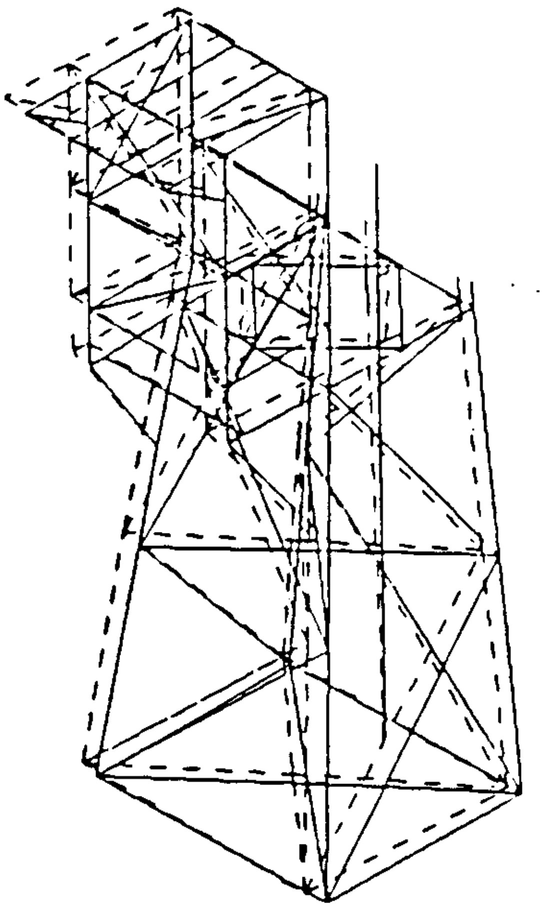
# 3D DEFLECTED ISO OF WP2/3
## Broadside Loading



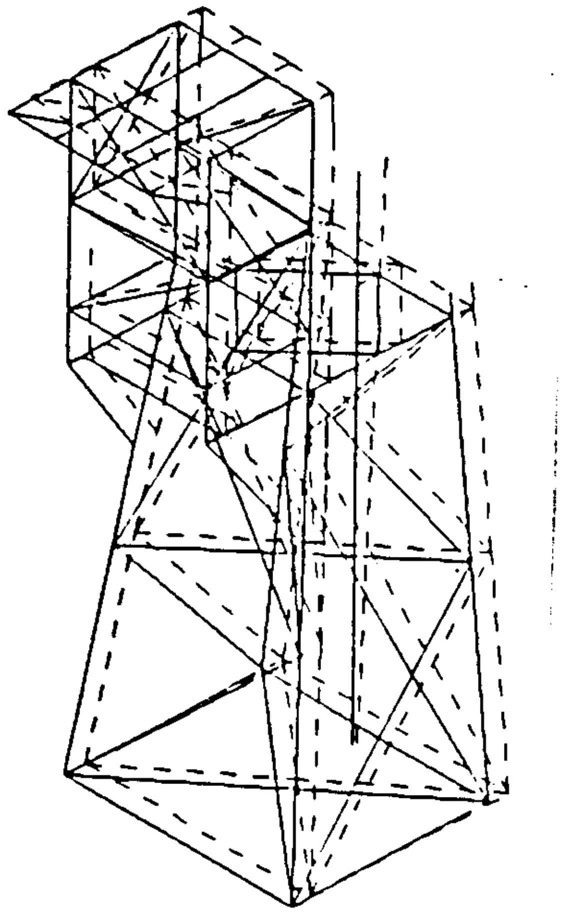
# 3D DEFLECTED ISO OF WP2/3
## End On Loading



# 3D DEFLECTED ISO OF WP4
# Broadside Loading

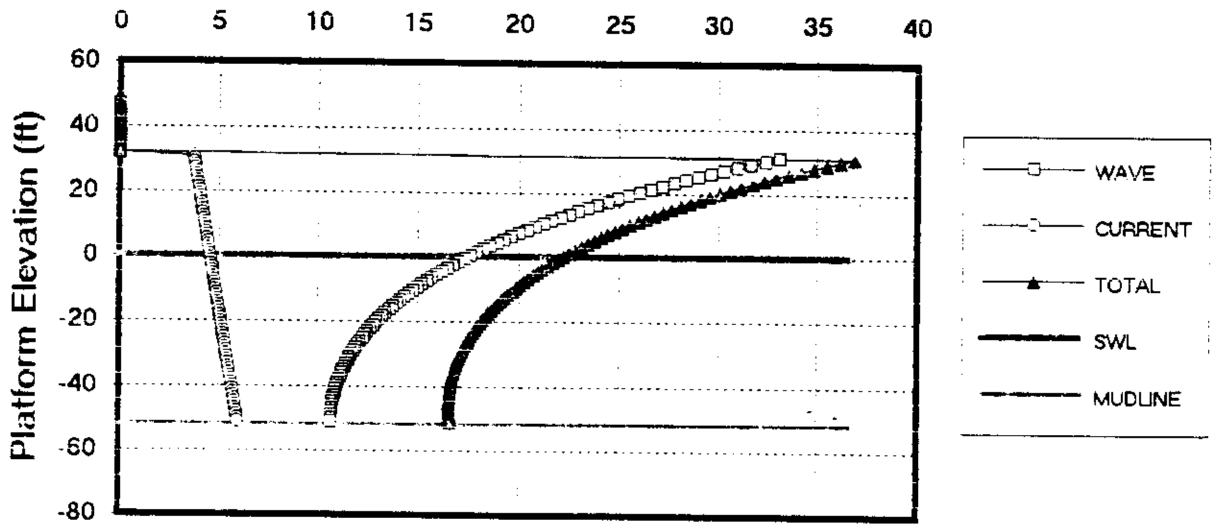


# 3D DEFLECTED ISO OF WP4
## End On Loading



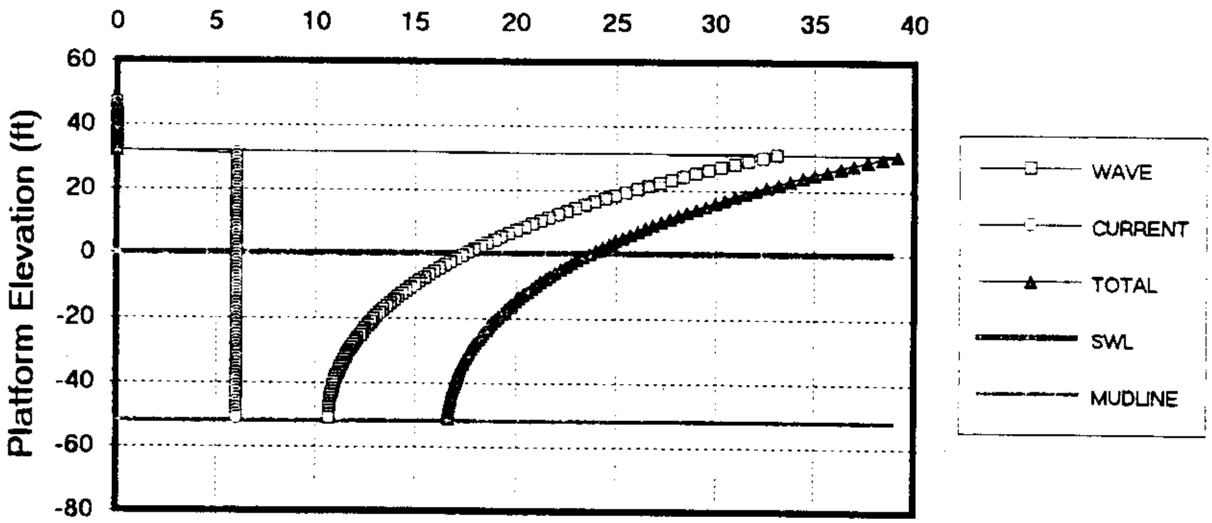
# WAVE KINEMATICS

Current with Mass Transport




Velocity (fps)

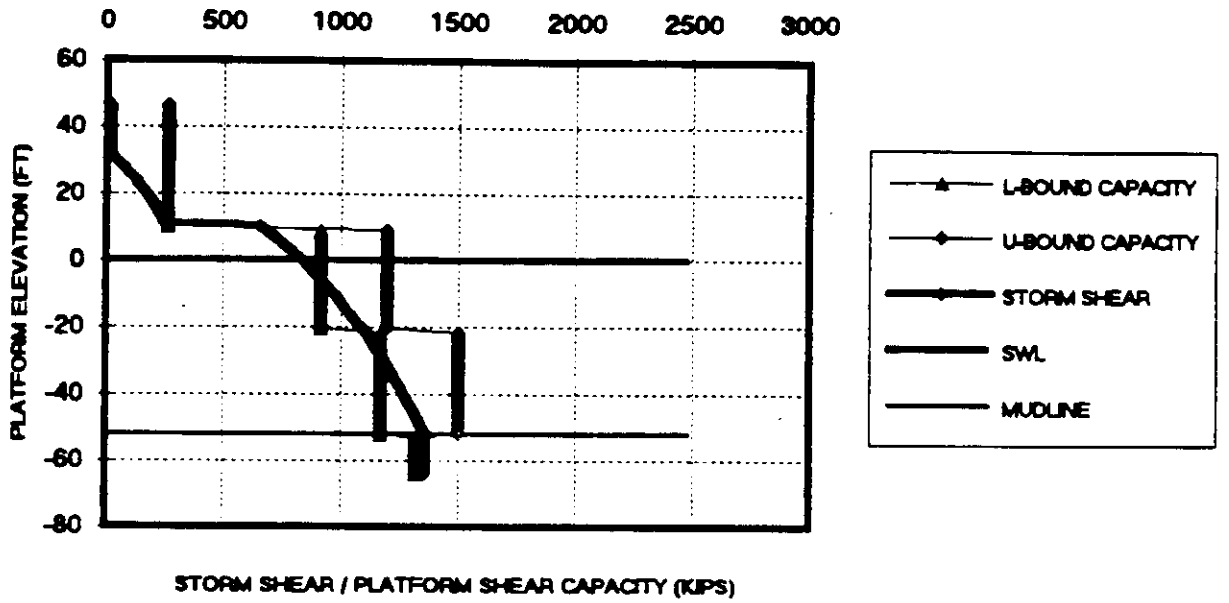
Constant Current



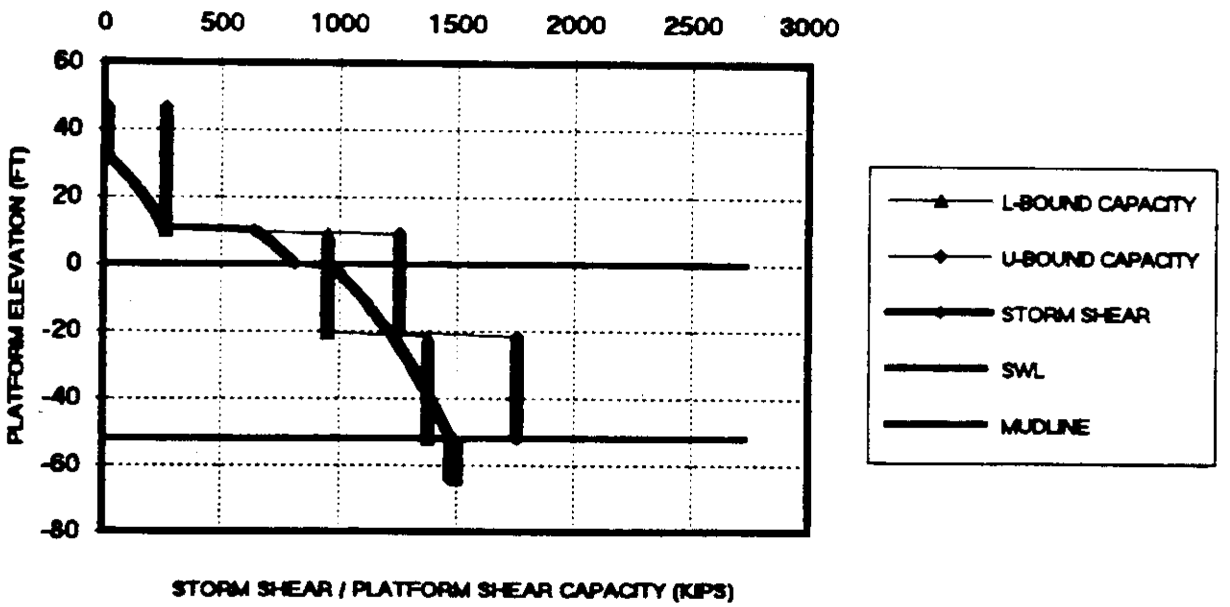

Velocity (fps)

# WP2/3 ANALYSES

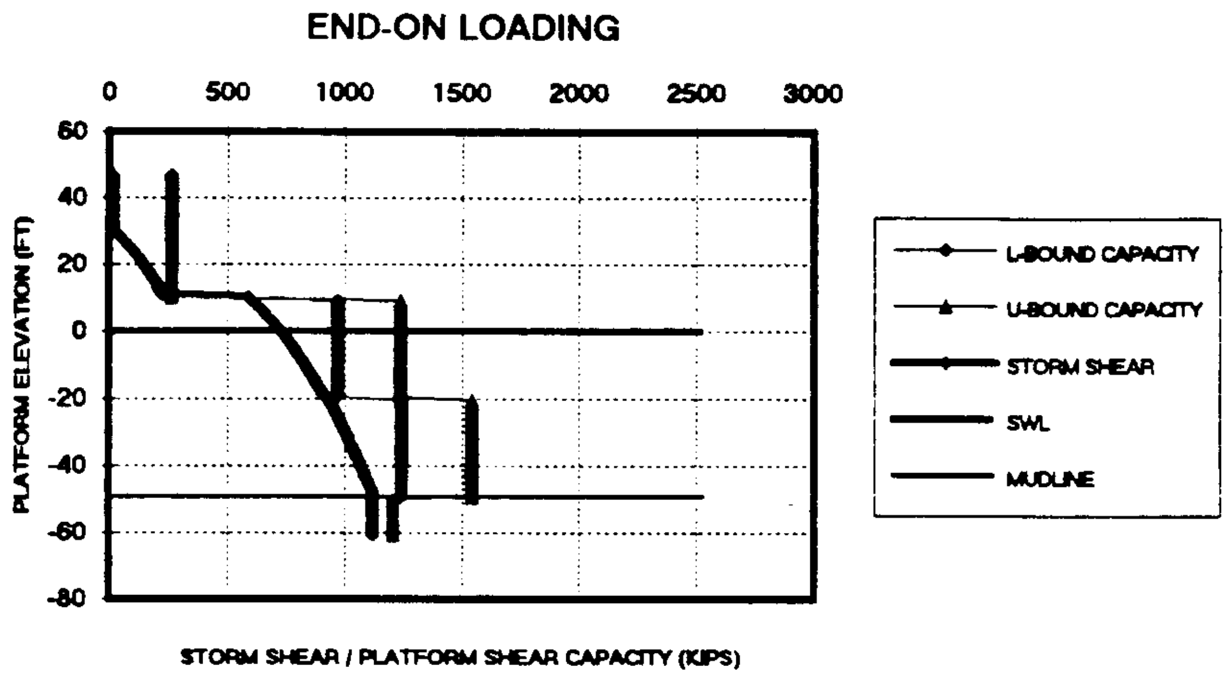
END-ON LOADING

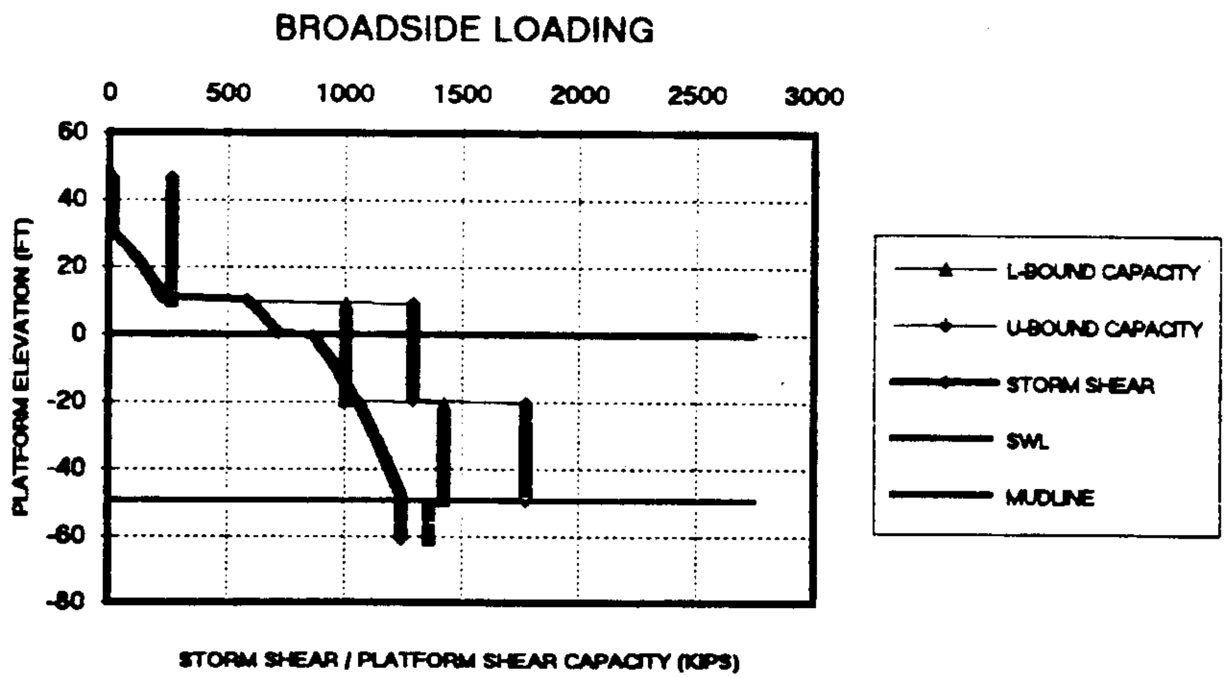



BROADSIDE LOADING

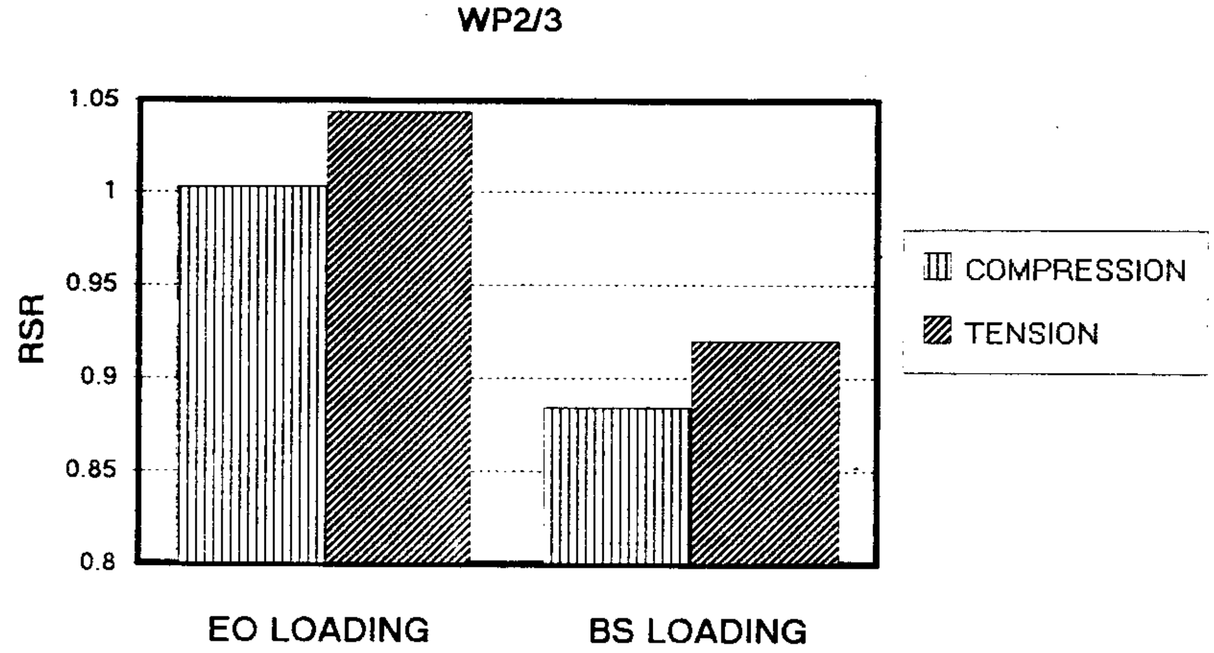


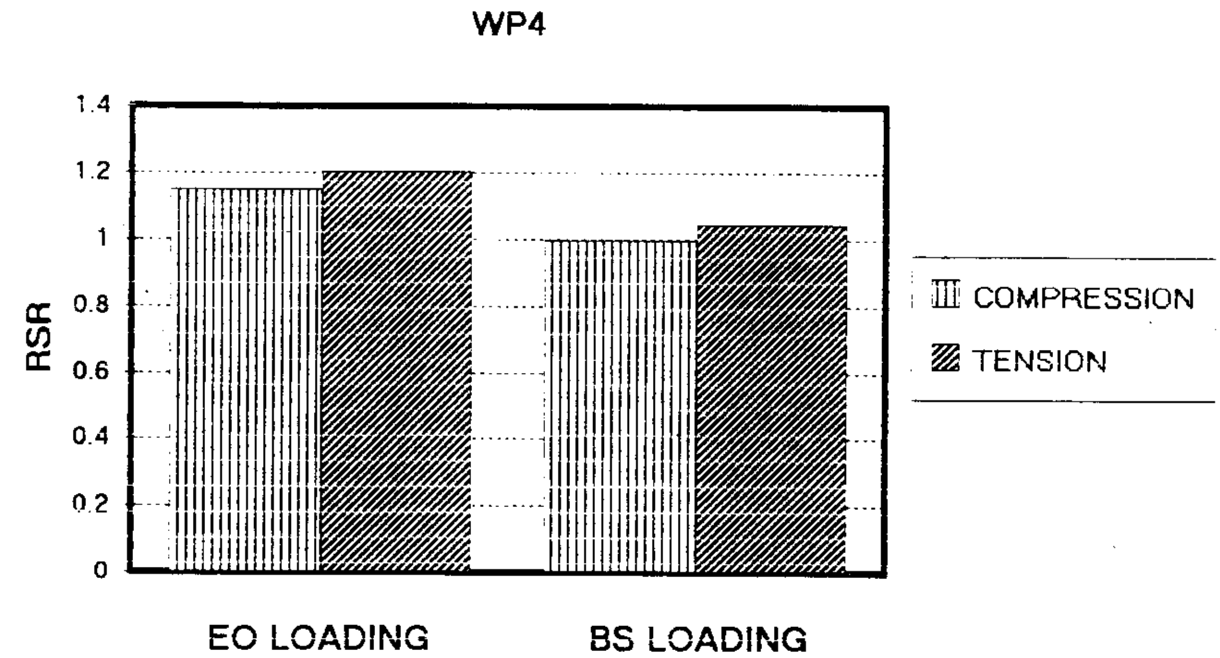
# WP4 ANALYSES



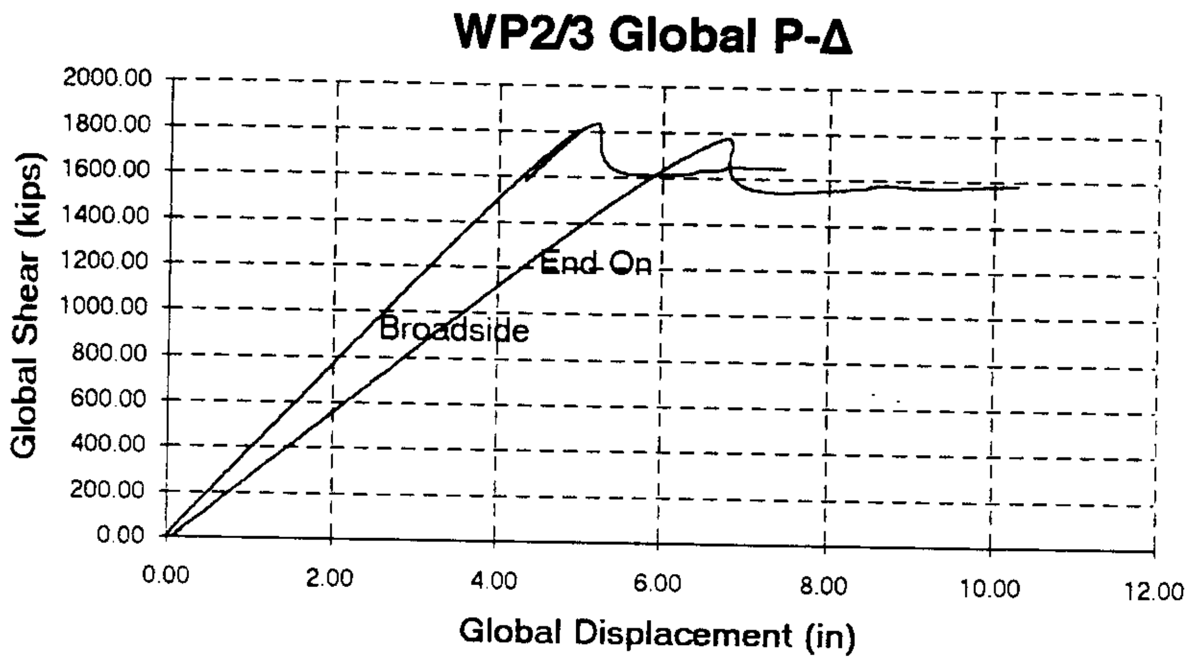



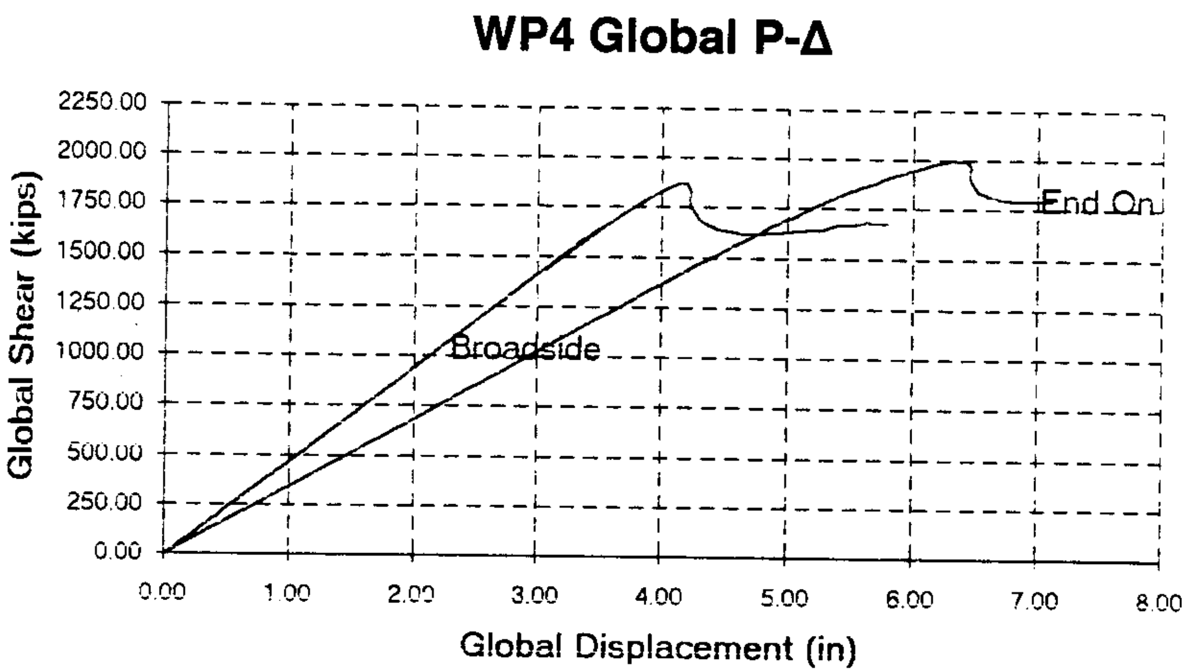
# AXIAL PILE CAPACITIES



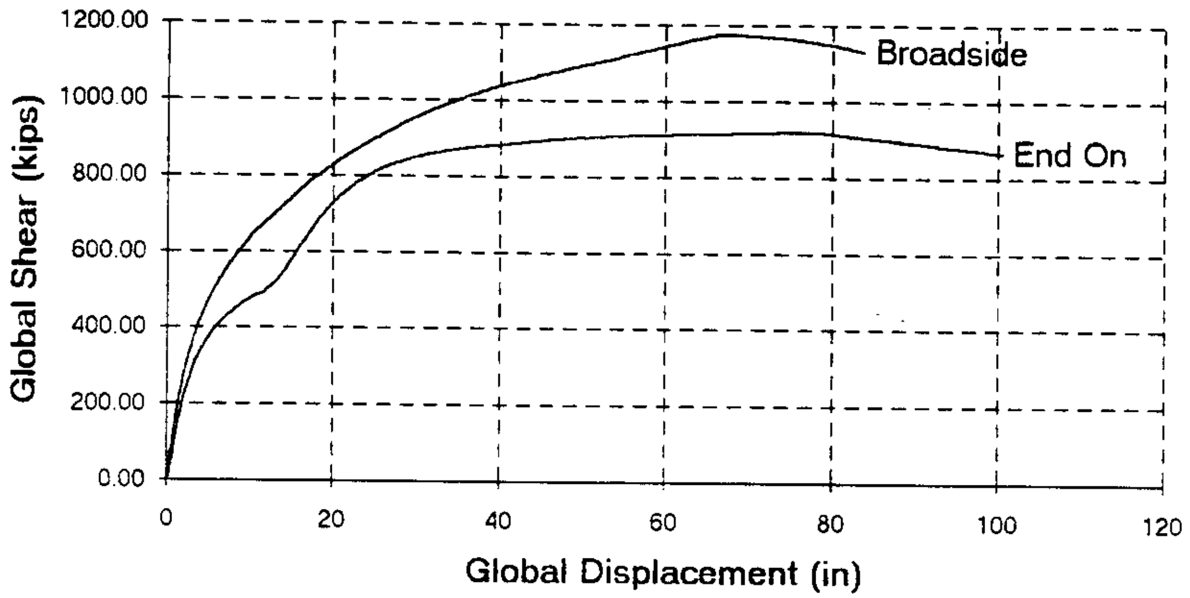
# COMPARISON OF USFOS PINNED PILE ANALYSES



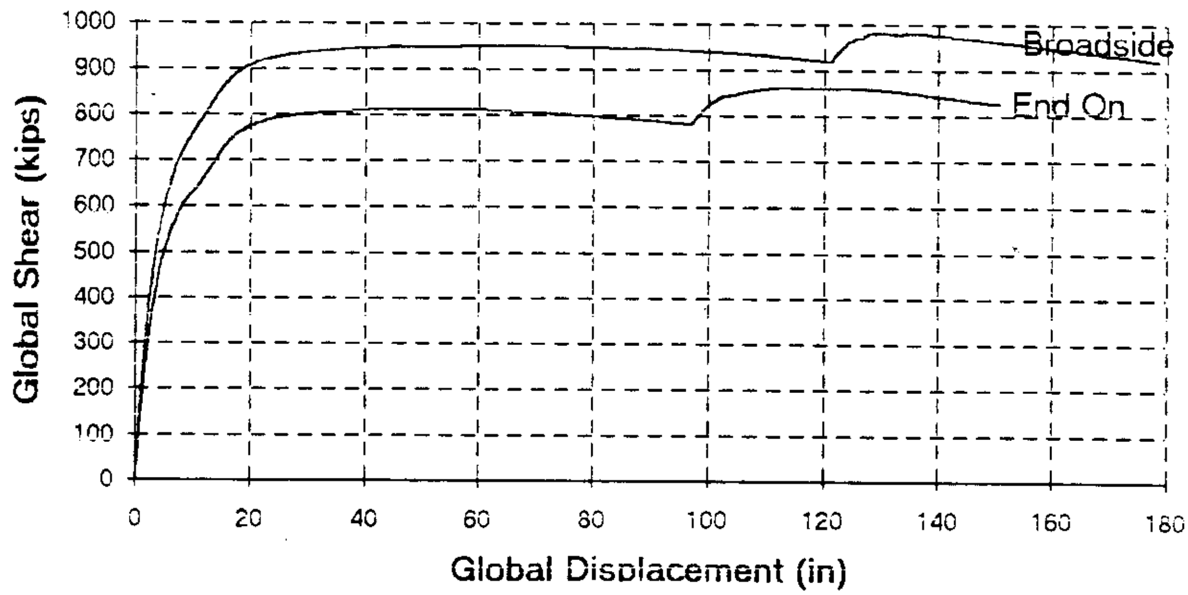



# COMPARISON OF USFOS SOIL SPRING ANALYSES

## WP2/3 Global P-Δ




## WP4 Global P-Δ



# Comparison of Storm Shears from Three Analyses

| | WP2/3 | | WP4 | |
|---|---|---|---|---|
| Analysis | EO | BS | EO | BS |
| StruCad | 1199 | 1299 | 1036 | 1088 |
| ULS 1 | 1218 | 1322 | 1000 | 1110 |
| ULS 2 | 1355 | 1474 | 1119 | 1244 |
| Usfos | 1141 | 1233 | 1036 | 1071 |

ULS 1 - ULSLEA with current mass transport

ULS 2 - ULSLEA without current mass transport

# CAPACITY COMPARISON BETWEEN ULSLEA AND USFOS

## Capacities

| | WP2/3 | | WP4 | |
|---|---|---|---|---|
| Analysis | EO | BS | EO | BS |
| ULSLEA | 1320 | 1521 | 1208 | 1363 |
| Usfos | 919 | 1178 | 861 | 984 |

## Reserve Strength Ratios

| | WP2/3 | | WP4 | |
|---|---|---|---|---|
| Analysis | EO | BS | EO | BS |
| ULSLEA | 0.974 | 1.032 | 1.080 | 1.096 |
| Usfos | 0.806 | 0.955 | 0.831 | 0.919 |

# COMPARISON OF RESULTS

## Dynamic Soil Spring Shear Capacities

| | Wells 2 and 3 | | Well 4 | |
|---|---|---|---|---|
| Loading | LF | Shear | LF | Shear |
| EO | 1.088 | 1242 | 1.205 | 1249 |
| BS | 1.289 | 1590 | 1.333 | 1427 |

## Andrew Storm Shear to Capacity Comparison

| | Time | Storm | | | Below Capacity | |
|---|---|---|---|---|---|---|
| | CDT | EO | BS | Total | R main | R tot |
| WP2/3 | 2100 | 1070 | 194 | 1087 | -13.8% | -12.5% |
| WP4 | 2000 | 392 | 817 | 906 | -42.7% | -36.5% |
| WP4 Br | 2100 | 430 | 752 | 867 | -47.3% | -39.2% |

# CONCLUSIONS

- Pile Foundation Capacities of Primary Concern
  - Low stiffness and inherent lack of redundancy is not compensated by lateral soil resistance
  - Piles form soft-story failure mechanism
  - Caissons prevent full formation of mechanism and induce pile pullout

- Likely Failure of WP2/3 Due To:
  - Additional caisson stiffness induces pile pullout quicker
  - Principal storm wave loadings parallel to end on structural orientation

- Likely Survival of WP4 Due To:
  - Lower storm shears due to lower water depth and wave breaking
  - Principal storm wave loadings roughly between end on and broadside directions

# PLANS

## October 1994 - April 1995

- Analyze Shell SP62 Platform Using Usfos
- Analyze Phillips/Mobil/Unocal/Exxon Platforms Using Usfos
- Analyze Dynamic Response of Single Well Caissons to Hurricane Andrew
- Perform Parametric Analyses
- Document Results

# PLANS FOR NEXT SIX MONTHS AND MEETING

- Finalizing the verification case studies
- Finalizing the work on Jacket-bays' lower and upper-bound capacities
- Finalizing the damaged and repaired element algorithms (corrosion, holes, joint cracks) and integrating them in SCREEN
- Finalizing and integrating the reliability analysis procedures in SCREEN
- Further automating the input and developing a graphical input check for SCREEN
- Finalizing SCREEN (completion, calibration, and revision) based on the latest research developments and sponsors' suggestions
- Next meeting proposed during April 1995